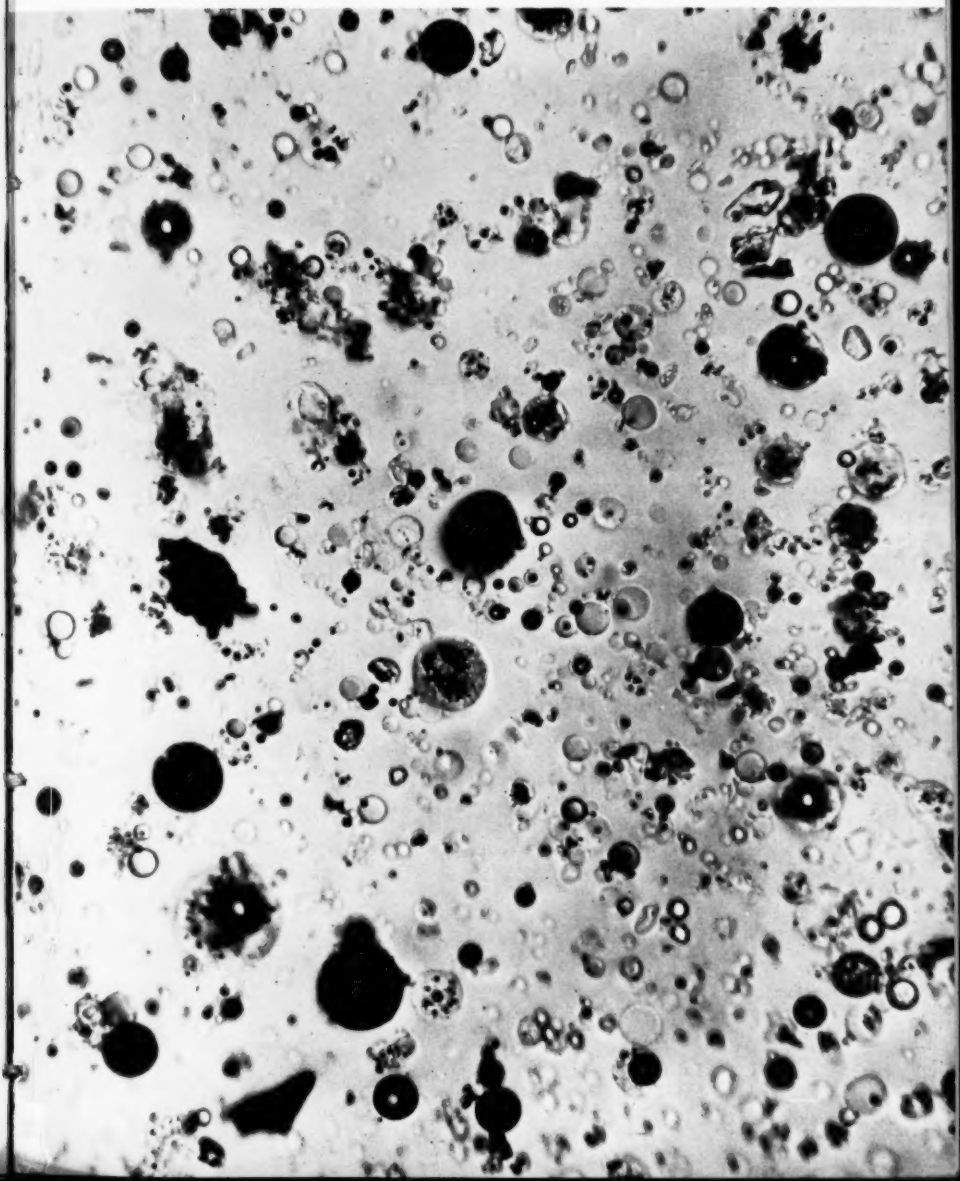
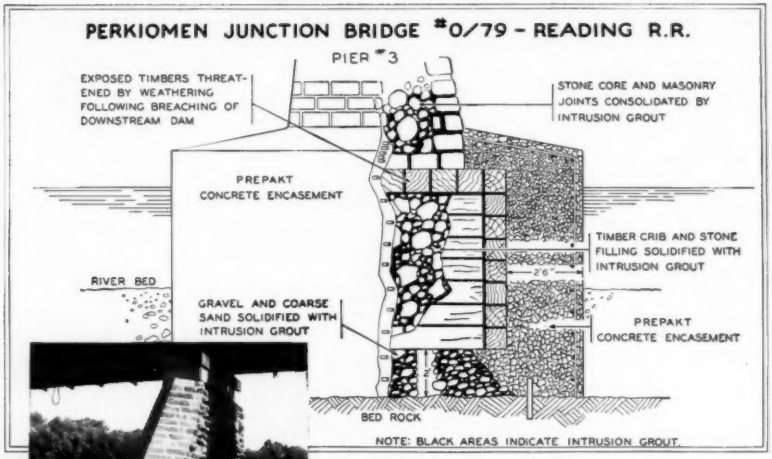
ACI News Letter

Vol. 21 No. 3 JOURNAL of the AMERICAN CONCRETE INSTITUTE November 1949




PERKIOMEN JUNCTION BRIDGE #0/79 - READING R.R.
PIER #3
EXPOSED TIMBERS THREATENED BY WEATHERING FOLLOWING BREACHING OF DOWNSTREAM DAM
STONE CORE AND MASONRY JOINTS CONSOLIDATED BY INTRUSION GROUT
PREPAKT CONCRETE ENCASEMENT
TIMBER CRIB AND STONE FILLING SOLIDIFIED WITH INTRUSION GROUT
RIVER BED
GRAVEL AND COARSE SAND SOLIDIFIED WITH INTRUSION GROUT
PREPAKT CONCRETE ENCASEMENT
BED ROCK
NOTE: BLACK AREAS INDICATE INTRUSION GROUT.

PREPAKT REPORTS ON . . .

STABILIZING AND STRENGTHENING OF BRIDGE PIERS

This old stone masonry bridge pier was founded on a timber grillage resting on gravel over bedrock. The upper timbers were exposed to air due to breaching of a down-stream dam and consequent lowering of the water level. To prevent deterioration, the alert Reading Railroad called on Intrusion-Prepakt to stabilize the pier *without interruption of railway traffic.*

As shown in the photograph and drawing, the pier base and cribbing were first encased and underpinned with Prepakt Concrete. The rock-filled crib, together with the coarse sand and gravel not removed for underpinning, was then consolidated with Intrusion grout. During the above operations, loose mortar was removed from the joints between masonry blocks, the joints filled with weather-resistant Intrusion mortar, and the interior of the masonry shaft Intrusion grouted.

Results:

1. A permanently stabilized pier capable of withstanding greatly increased loads.
2. No interruption of traffic.
3. An order to repeat on another pier of the same bridge.

THE CONCRETE WITH PREPAKT EXTREME DURABILITY

INTRUSION-PREPAKT, INC. THE PREPAKT CONCRETE CO.

CHICAGO · TORONTO CLEVELAND 14, OHIO PHILADELPHIA

ACI News Letter Contents

November 1949

On the Cover—Microphotograph (approx. 750X) of a typical fly ash showing the large percentage of spherical glass particles which is the predominant material in fly ash. It is being used for its pozzolanic properties by the U. S. Bureau of Reclamation in Hungry Horse Dam.

Courtesy
U. S. Bureau of Reclamation

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How far have we come?

One year ago, the ACI Board of Direction, recognizing the Institute's growth and stature and means and opportunities for increasing both, adopted a special committee report setting up ACI goals and giving recommendations for attaining them.

How far has ACI come after one year toward those goals? To realize the progress made, it is best to review the major goals set before the Institute.

1. An increase in membership to approximately double the present membership in ten years and ultimately to include substantially all persons who would be usefully served by the Institute.

In the year Oct. 1, 1948 to October 1, 1949 membership increased from 4288 to 4781, a total of 493—11.5 percent. It is probably too much to expect the membership to continue to increase at the same percentage rate. But it is not unreasonable to expect that (assuming the peaks and valleys of the general economy are not in unfavorable contrast) the membership increase will continue at slightly more than last year's, say about 500 per year as a conservative figure. At that rate, the goal of double the membership of October 1, 1948—8576—should be reached well within the ten-year period, probably about seven and a half or eight years from now.

It is only through continued *Member Consciousness* that this goal is possible. ACI would then be reaching more of those who can be served usefully by ACI, and in turn they will be serving fellow members.

2. Increase in the number of and improvement in the technical quality of papers with particular emphasis on a larger number of comparatively short papers.

Continued on p. 4

More and better papers with the emphasis on short papers is a goal not easily attained, but progress has been made. In the past year the JOURNAL has published 45 papers and reports averaging 12.9 pages each. Actual lengths were from 4 to 44 pages. Editorial activity in soliciting more diversified papers is not yet apparent but it is hoped it will produce results in the next few months.

3. Broaden the scope of technical papers to include more of those describing methods of construction, architectural features and details of techniques successful in obtaining attractive and durable structures.

There is evidence that progress is being made toward this goal too, although the complete realization of it will be difficult. Construction men are notably reticent about their work as are many of the product manufacturers. JOURNAL papers in the past year have fallen into the following general classifications: construction and construction practices, 20 papers; products manufacture, 2; research, 17 and design, 6. The ideal arrangement for which the staff strives is about one article from each of these broad classifications in each issue of the JOURNAL.

4. Increase in the quantity and scope of technical committee reports.

Committee activity, which in time will result in more reports on technical subjects has increased in the past year. Especially of note is the activation of Committee 323—Prestressed Reinforced Concrete under the chairmanship of A. E. Cummings and Committee 325—Structural Design of Concrete Pavements for Highways and Airports under the chairmanship of L. W. Teller. Committee 318—Standard Building Code has been reorganized under the chairmanship of Chester L. Post and has resumed its review of new developments likely to affect code requirements. F. N. Menefee, chairman of Committee 711—Precast Floor Systems for Houses is spark-plugging a half session at the Chicago convention aiming to bring into focus and under discussion important stumbling blocks common to the work of Committees 208, 318, 323, 324 and 711.

A new standard of Committee 616 on the use of cement paints was adopted during the year and Committee 805—Recommended Practice for the Application of Mortar by Pneumatic Pressure has a report which is expected to be ready for publication in the near future.

Other committees are active on matters less spectacular but equally important.

5. Increase in the technical value and appeal of convention sessions.

6. Increase in the number of and in the importance of discussions of technical papers and committee reports.

Little progress can be reported on items 5 and 6 although the 1950 convention will feature considerably more material in short papers covering seven sessions instead of the usual five, by having four of the sessions in concurrent pairs in two periods.

7. Development of a policy of taking the Institute to the members, such as by increasing the number of regional meetings and the holding of local conferences.

The Institute IS being taken to the members. Anticipating the committee's recommendations the first regional meeting was held successfully in Birmingham, Alabama, in the fall of 1947. This year the New England Regional Meeting in Boston, November 10 and 11, 1949 scheduled a top-notch program. Also in the past year and a half Secretary Whipple has made several trips in connection with convention arrangements and has availed himself of the opportunity to meet with groups along the way to get acquainted and to tell about ACI.

The remainder of the report dealt with many elements of the work of the Institute and the staff to complete these assignments. This work included *an extension to the ACI 10-Year Index, an improvement in JOURNAL layout and readability, additional activity by technical committees, improved convention sessions and programs, enlargement of staff, and increased scope and activity of the Institute staff.*

One year is a short time for achieving such an extensive list of goals, but some of those goals have been reached and others are near completion.

Staff activities have been enlarged with the addition of two engineers familiar with editorial work. This has made possible the editing and rewriting of more papers for publication, an enlargement of the Current Reviews and Letters from Readers sections of the JOURNAL, a more complete indexing of JOURNAL literature, with the ACI 20-Year Index rolling off the presses in the spring of 1950, and a chance to work more closely with committees in organizing and preparing reports.

Last, but not least, the ACI staff after December 1 will be in still modest but more spacious quarters conducive to increased efficiency and output which is not possible in the present over-crowded office.

This report to Members will show that the goals set have not yet been achieved but progress is being made. A goal reached too easily is a goal set too low—the reach should exceed the grasp as a poet has said. We hope that ACI's 1948 goals will be reached sooner than expected—and their scope extended. With the continued cooperation of the Membership they will be.

Members of the Institute may have other suggestions for achieving the desired goals. They would be welcomed by the Institute staff.

Who's Who This Month

Arthur P. Clark

Arthur P. Clark, Research Associate for the American Iron and Steel Institute at the National Bureau of Standards, Washington, D. C., is no stranger to JOURNAL readers. He continues the discussion of bond in "Bond of Concrete Reinforcing Bars," p. 161. His earlier papers in the JOURNAL include "Highlights of the Development of Reinforced Concrete in the Study of Bond," Feb. 1948 and "Comparative Bond Efficiency of Deformed Concrete Reinforcing Bars," Dec. 1946.

He is the co-author of the design book *Useful Data on Reinforced Concrete Buildings* and developed and patented a completely shop fabricated unit of reinforcing bars for beams known as "Corr-Bar Units."

After graduating from the University of Michigan in 1903 with a B.S. in civil engineering, Mr. Clark spent 3½ years in the bridge department of the Pere Marquette Railroad. He has been associated with reinforced concrete engineering and reinforcing distribution studies since 1906, first with the Corrugated Bar Co. of St. Louis and then continued with the company when it consolidated with Kalman Steel Co. and was later purchased by Bethlehem Steel Co. Mr. Clark has been at the Bureau of Standards since 1944.

Mr. Clark has been an ACI Member since 1924 and is a member of Committee 208, Bond Stress, and is also a member of A.S.C.E., A.S.T.M. and an associate member of the Highway Research Board.

J. John Brouk

J. John Brouk, Vice-President, Precast Slab and Tile Co., St. Louis, Mo., in his first JOURNAL contribution, presents information on "Perlite Aggregate: Its Properties and Uses," p. 185. He has been active in the research and development of lightweight aggregate, especially perlite.

After graduating with a degree in ceramic engineering from the University of Illinois in 1938, Mr. Brouk was associated with the Hydraulic Press Brick Co. in the haydite concrete aggregate department. He was manager of the department until 1942.

He served in Naval Research 1942-46, and was made vice-president of Precast Slab and Tile Co. in 1946. He helped design concrete for the U. S. Maritime Commission concrete barges and has developed a lightweight insulating fire brick bonded with portland cement.

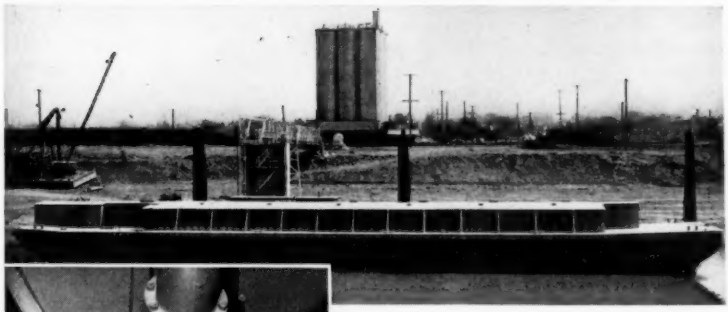
Francis R. Mac Leay

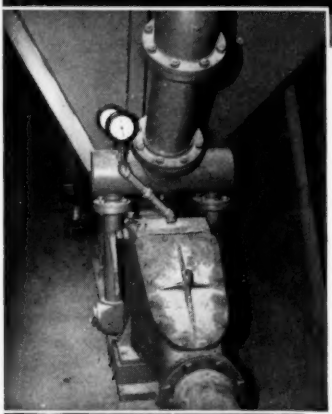
"Thin Wall Concrete Ship Construction," p. 193, originally presented at the 1946 ACI convention in Buffalo, was revised by Francis R. Mac Leay prior to his death in July. At the time of his death, Mr. Mac Leay was chief engineer and general project manager for John A. Johnson and Sons, Brooklyn. He had previously been chief engineer for the Corbetta Construction Co., Inc., of New York and Chicago.

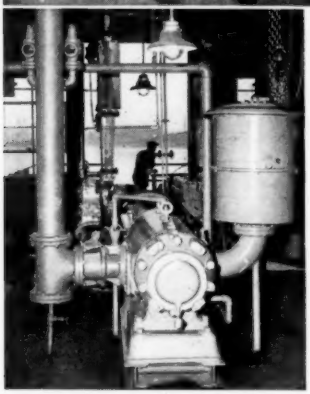
During the war, Mr. Mac Leay was in charge of concrete ship construction for the U. S. Navy at the College Point Shipyard, so was well acquainted with the problems involved in thin wall concreting.

An ACI Member, Mr. Mac Leay was associated with various building projects as well as being a lecturer, writer and teacher. He was the author of *Practical Analysis of Rigid Frames* and *Manual of Structural Planning and Design* and numerous articles on concrete in engineering magazines.

A graduate of Northeastern University in civil engineering, Mr. Mac Leay had







UNLOADING CEMENT FROM BARGES AND CONVEYING TO STORAGE

. . .

FULLER-KINYON PUMP
AND
FULLER COMPRESSOR

Fuller-Kinyon Pumps can be used for many purposes, applied in many different ways.

The installation illustrated shows how the Fuller-Kinyon Conveying System, of the stationary type, is applied to the unloading of bulk Portland cement from river barges. The Fuller-Kinyon Stationary Pump, installed in the hold of the barge, (photo left) unloads and conveys the cement direct, in one operation, to storage silos on shore. The cement is delivered to the pump by a drag scraper.

Air for the conveying system is furnished by a Fuller Rotary Single-stage Compressor, (photo lower left) installed on a captive barge alongside of the dock. Capacity of compressor 1552 c. f. m., at 35-lb. pressure. Air where and when needed, and at the right pressure to do the work most efficiently and economically.

If you have what you consider a "different" or "difficult" conveying problem, it may be that the Fuller-Kinyon System will solve it for you. It will cost you nothing to find out. Our engineering department is at your service . . . no obligation whatsoever.

FULLER COMPANY, CATASAUQUA, PA.

Chicago 3 - 120 So. LaSalle St.
San Francisco 4 - 420 Chancery Bldg.

P-109

been a consulting engineer, construction supervisor for the Philadelphia Transit Commission, and was associated with various New York architectural firms. As chairman of the New Rochelle Housing Authority, he supervised two major slum clearance projects.

As a structural consultant, designing engineer or concrete supevisor, Mr. Mac Leay had worked on 20 public housing projects in New York and many bridge projects, including the George Washington, Triborough, Bronx-Whitestone and Henry Hudson bridges in New York, the Rip Van Winkle bridge in upper New York State and bridges over the Delaware and Susquehanna Rivers.

He had also participated in the construction of the Museum of Modern Art in New York, the Virginia State Library and Supreme Court Building, the American College at Athens, Greece, the American College for Girls at Hellenica, Greece, and Butler University buildings at Indianapolis.

He was a member of A.S.C.E., the Boston Society of Civil Engineers and the Society of American Military Engineers. He was vice-president and former president of the Consulting Structural Engineers Society of New York and former president of the Westchester County Architectural and Engineering Alliance.

Harry F. Thomson

Harry F. Thomson, consulting engineer, St. Louis, Mo., and ACI Vice-President, presents "Specifications Should Be Realistic" on p. 205, which reflects observations from contacts with architects, engineers, public agencies and contractors with reference to concreting over a span of 20 years.

Mr. Thomson, who has been prominently connected with the ready-mixed concrete industry for many years, has been active in Institute affairs since 1928. He was elected to the ACI Board of Direction in 1939, serving till 1940. He was Director-at-Large, 1942-47, Director, 1948 and elected Vice-President in 1949. Mr. Thomson was a member of the Publications Committee, 1940-41 and has served on other ACI technical and administrative committees. He has made several contributions to the JOURNAL as well as to other technical publications.

After graduating from Washington University, Mr. Thomson served as instructor and research worker at the Massachusetts Institute of Technology for 4 years. Following connections with manufacturers in St. Louis, New York and Birmingham, Ala., he became associated with the General Material Co., St. Louis, in 1927, one of the pioneer producers of ready-mix concrete in the Middle West. He continued with that company as vice-president and subsequently president until 1948. Since then he has acted as consulting engineer, specializing in ready-mix plant and merchandising problems.

Mr. Thomson served as president of the National Ready Mixed Concrete Assn. and has been chairman of that organization's Technical Problems Committee for many years. He is a former national director of A. S. C. E. Mr. Thomson has also been active in civic affairs, having served as director of the St. Louis Chamber of Commerce and was one of the organizers of the Metropolitan Plan Assn. of St. Louis.

Reviewer of Italian literature needed

For greater coverage in "Current Reviews" of foreign engineering publications, a volunteer reviewer is needed for the Italian-language magazine *Giornale del Genio Civile* published in Rome. A similar prompt response to that elicited by the September appeal for Dutch reviewers will add to our pages cognizance of another segment of foreign literature. Drop a note to the Secretary indicating your willingness to serve and further details will be forthcoming.

BALDWIN
CEMENT and CONCRETE COMPRESSION TESTING MACHINE

—300,000 lb. CAPACITY
—NEW DESIGN
—NEW FEATURES

Meets all A.S.T.M. requirements, provides for testing concrete cylinders up to 8″ x 16″, and also is equipped for testing concrete building blocks up to 12″ wide x 18″ long.

Single ram construction, increased ram stroke, and fast operation facilitate alternate testing of cylinders and blocks.

Write to us for further details.

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The Baldwin Locomotive Works, Philadelphia 42, Pa., U. S. A. Offices: Birmingham, Boston, Chicago, Cleveland, Houston, New York, Philadelphia, Pittsburgh, San Francisco, Seattle, St. Louis, Washington. In Canada: Peacock Bros., Ltd., Montreal, Quebec.

We Want to Know—

ACI Members will recall that the membership classification forms circulated last year contained a question—"What suggestions have you for the Technical Activities Committee as to papers or reports you'd like to see published; subjects you'd like to see discussed?" This is the third group of such questions published, the first group appearing in the September *News Letter*.

One thing that will undoubtedly develop from the many replies is a special session at the Institute's convention next February at which a panel of "experts" would head the discussion.

Meanwhile the ACI JOURNAL will publish anonymously a good many of these questions and suggestions. JOURNAL publication is intended to bring responses which may be the answer or partial answer to questions asked. Readers are not only invited to answer those questions appearing in print, but to send other suggestions and queries as well. Some of these questions have been answered in past ACI literature and will be covered more thoroughly in future issues of the JOURNAL Letters from Readers sections.

—Editor

Air retained after set

What is the effect of different types of slip-forms and concrete placing machines on the percent of air retained in air-entraining concrete after final set?

How?

I am particularly interested in articles on "How to do it"—both in construction and design.

Thank you

I think the Institute is covering the field of concrete quite well, although for myself I am more interested in articles pertaining to advancements in structural theories and design.

Steel forms

Can fixed steel plates be economically substituted for formwork?

Pro and con

The ACI JOURNAL carries a lot of warranted prestige and influence. Whenever a controversial article is published it may be accepted as gospel before it has been subjected to enough proof. Why not some simultaneous articles on both sides of such a subject in the same issue of the JOURNAL, or at least a summarizing brief of the opposite viewpoint by a qualified spokesman?

Behavior of structure

More emphasis should be given to the behavior of the actual structure. American engineering needs more knowledge and discussion of actual structural performance under working conditions.

Construction details.

I would like to see more papers on construction details and how they are done on large projects.

Wind forces

What is the effect of wind and other lateral forces upon arches and other curved members?

Coloring

What is the best method of coloring concrete?

Pavement problems

Research data on the cause of buckling of concrete pavements, as well as pumping action, joints, cement stabilized bases and related subjects would be appreciated.

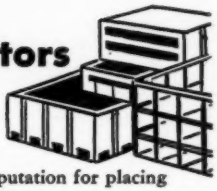
Form pressures

I would like to see form pressures discussed.

Continued on p. 18

Profits for Everyone USING or MAKING Concrete with PROTEX AEA

contractors



Contractors cut costs through . . .

- **Increased workability and placeability**
- **Faster finishing with Protex because of reduced surface water, finishing immediately after placing and less troweling.**

Contractors using Protex can establish a reputation for placing higher quality and better appearing concrete.

block and pipe plants

Block and pipe plants cut costs through . . .

- **Increased production**
- **Less green breakage of blocks**
- **Less breakage of pipe during stripping**
- **Increased material yield**

Provides an opportunity to increase sales by establishing a reputation for selling higher quality and better appearing blocks and pipe.

ready-mix, pre-mix, etc.

Manufacturers of concrete, such as Ready-Mix and Pre-Mix, cut costs through . . .

- **Saving in material**
- **Reduced segregation**
- **Faster discharge because of increased flowability**

Provides an opportunity to increase sales by establishing a reputation for selling higher quality and better appearing concrete.

Protex is being widely used by: U. S. Bureau of Reclamation, U. S. Corps of Engineers, Bureau of Public Roads, State and City Governments.

● **LOW COST PROTEX DISPENSER... *AUTOMATICALLY* DISPENSES PREDETERMINED AMOUNT OF AEA.**

FREE BOOK ON AIR ENTRAINMENT

AUTOLENE LUBRICANTS CO.
Industrial & Research Division, Denver 9, Colorado

Please send me your book, "Facts on Modern Placement of Concrete Through Air Entrainment."

*Name*_____

*Address*_____

Joshua L. Miner

Joshua L. Miner, retired vice-president of the Lumnite Division of Universal Atlas Cement Co. and ACI Member from 1922 to 1947, died recently in Plainfield, N. J.

Mr. Miner joined the cement company in 1922 and in 1924 became identified with a subsidiary company, the Atlas Lumnite Cement Co., as manager of sales, production and research. In 1937 he was elected director and vice-president of that subsidiary. When the subsidiary became the Lumnite Division of Universal Atlas Cement Co., he was elected vice-president of the company, which position he held until his retirement in 1947.

After graduating from Lafayette College in 1903, Mr. Miner began work as a chemist, spent 10 years as manager of the Pittsburgh Testing Laboratory in Dallas and New York, and in 1922 became associated with Universal Atlas.

He was a member of A.S.T.M. Committee C-1 for more than 30 years, served on the society's executive committee, and was author of several papers on cement and concrete.

During World War I, Mr. Miner served in a supervisory capacity with the Chief of Inspection, Purchase, Storage and Traffic, General Staff, U. S. Army. During World War II he was chairman of the Technical Advisory Committee of the Cement, Lime and Gypsum Division of the War Production Board.

Since the introduction of calcium-aluminate cement in this country, Mr. Miner had been largely responsible for the development of its manufacture and use. In recent years he had directed the research and field work which has resulted in the growing use of calcium-aluminate cement for refractory service and corrosion-resistant installations.

Charles R. Gow

Word has been received of the death of Col. Charles R. Gow, foundation engineer. He invented the Gow caisson pile and the Gow method of test boring which led to specialized foundation work by the firm bearing his name.

Col. Gow was a director of ACI 1916-19, vice-president 1920-21, and winner of the second Wason Medal for the most meritorious paper at the 1917 convention, "History and Present Status of the Concrete Pile Industry."

The Charles R. Gow Co. became a subsidiary of Raymond Concrete Pile Co. in 1922, with Col. Gow remaining as president, 1922-29. He continued as consultant and advisor until 1932. During 1921-23, he was president of the Associated Industries of Massachusetts. He was appointed the first Professor of Humanics at the Massachusetts Institute of Technology in 1928, and Postmaster of Boston 1929-30. In 1930 Col. Gow was elected president of Warren Brothers Co. and was chairman of the board at the time of his death.

Charles A. Ellis

Charles A. Ellis, designing engineer of the Golden Gate Bridge and ACI Member, died recently in Evanston, Ill.

Widely known in engineering circles, Mr. Ellis also designed the Montreal Harbor Bridge and many railroad and highway bridges in the United States. Head of the Division of Structural Engineering at Purdue University from 1934-1946, he had been professor emeritus since then. Previously he had been associate professor at the University of Michigan and the University of Illinois.

Mr. Ellis was connected with the American Bridge Co. 1902-1908, vice-president of the Strauss Engineering Corp. 1921-1932, and in private practice 1932-1934. He had been a lecturer in civil engineering at the Northwestern University Technological Institute since 1946.

He wrote several books on civil and structural engineering as well as many articles in engineering publications. He was also a member of A.S.C.E. and the American Railway Engineering Assn.

Honor Roll

February 1, to September 30, 1949

The top 66 names on our Honor Roll represent 22 states and 10 countries outside of the United States. If your state or country is not represented among this group why don't you act promptly to recommend some acquaintance for membership in the American Concrete Institute?

We'd like to see every state and country listed in the top flight of member getters before the current Honor Roll ends January 31, 1950.

Newlin D. Morgan (Ill.)	42½
Newlin D. Morgan, Jr. (Wyo.)	26
F. N. Menefee (Mich.)	11½
R. H. Sherlock (Mich.)	7
Jaime Alberto Mitrani (Cuba)	6
Jose Antonio Vila (Cuba)	5
H. J. Gilkey (Ia.)	4½
James A. McCarthy (Ind.)	4
J. Vicente Orozco (Mexico)	4
John J. Hogan (N. Y.)	3½
Frank E. Richart (Ill.)	3½
Thomas E. Stanton (Calif.)	3½
Raymond E. Davis (Calif.)	3
H. D. Jolley (Ill.)	3
Barton Jones (Puerto Rico)	3
J. W. Kelly (Calif.)	3
Frederic F. Mavis (Pa.)	3
William T. Neelands (Ga.)	3
Leon Venegas (La.)	3
Stanton Walker (D. C.)	3
Harold S. Carter (Utah)	2½
R. A. Caughey (Ia.)	2½
Roger H. Corbetta (N. Y.)	2½
H. F. Gonnerman (Ill.)	2½
A. J. McElrath (Tenn.)	2½
Paul W. Abeles (England)	2
A. Amirikian (Md.)	2
J. F. Baxter (N. Y.)	2
Sterling Lowe Bugg (Fla.)	2
W. S. Cottingham (Wis.)	2
John G. Dempsey (N. Y.)	2
Ambrosio R. Flores (Calif.)	2
Zorislav Franjetic (Argentina)	2
Henry E. Griset (N. C.)	2
Robert J. Hansen (Mass.)	2
V. P. Jensen (Calif.)	2
Emory E. Johnson (S. D.)	2
R. R. Kaufman (Ohio)	2
Karl W. Lemcke (N. Y.)	2
William S. Moore (Ind.)	2
William D. Nowlin (Va.)	2
Luis Paredes-Manrique (Colombia)	2
Luis A. Pietri-Lavie (Venezuela)	2
Dan H. Pletta (Va.)	2
Thomas C. Shedd (Ill.)	2
G. E. Troxell (Calif.)	2
George Winter (N. Y.)	2
Roberto Barillas F. (Guatemala)	1½
S. J. Chamberlin (Ia.)	1½
Jacob J. Creskoff (Pa.)	1½
H. C. Delzell (Ill.)	1½
A. H. Douglas (Canada)	1½
Adly W. Gindy (Egypt)	1½
Adli Milik Hanna (Egypt)	1½
Elmo C. Higginson (Colo.)	1½
Henry L. Kennedy (Mass.)	1½
Robert L. Mauchel (N. Y.)	1½
Sam McCluer (Ga.)	1½
H. J. McGillivray (Fla.)	1½
Harmon S. Meissner (Colo.)	1½
Carl A. Menzel (Ill.)	1½
Edward W. Scripture, Jr. (Ohio)	1½
Roy T. Sessums (La.)	1½
Bertrand H. Wait (N. Y.)	1½
Calvin T. Watts (La.)	1½
Frank W. Wheeler (Va.)	1½
Jerome O. Ackerman	1
William F. Anderson	1
Samuel Baker	1
J. A. Bakker	1
Charles F. Ball	1
John H. Bateman	1
Donald C. Beam	1
Oscar Benedetti	1
H. J. Bezette	1
Robert F. Blanks	1
Hans H. Bleich	1
Ernest M. Bowles	1
R. W. Brandley	1
Carlos Mora Bravo	1
Milton Brumer	1
R. Y. Bush	1
Adraino Carbone	1
D. R. Cervin	1
R. V. Chate	1
H. Clare, Jr.	1
E. F. J. Clark	1
H. W. Comack	1
R. E. Copeland	1
Alirio Cornejo	1
Hendrik Cornelissen	1
Leo H. Corning	1
G. H. C. Crampton	1
Ray A. Crysler	1
Jose A. Cuevas	1
Harmer E. Davis	1
W. Dewoody Dickinson, Jr.	1
Peter J. Doanides	1
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Jiri Stork 1
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Manabu Tanaka 1
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T. W. Thomas 1
I. L. Tyler 1
Oscar J. Vago 1
Joseph J. Waddell 1
John M. Wells 1
J. G. Welsh 1
Donald K. Woodin 1

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Michel Bakhoum — J. W. McBurney
C. Harold Barcus — Robert B. McCalley, Jr.
Charles H. Barker — T. R. McCullough
Hugh Barnes — Jack Robert Miller
Frank Baron — Sylvester Morabito
J. F. Barton — Wm. Morris
J. E. Bates — William H. Munse
Aaron H. Bauman — S. Neatwait
E. W. Bauman — Philip M. Noble
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A. H. Brewer — Chester B. Palmer
Harold W. Brewer — Douglas E. Parsons
A. R. Brickler — Robert J. Perrich
Robert G. Burnett — George Nelson Perry
Henry B. Campbell — Alan H. Pilling
J. A. Carrick — Warren Raeder
Napoleon V. Campomanes — Irvin S. Rasmusson
W. R. Chandler — F. V. Reagel
Arthur P. Clark — Allan M. Reese
Leo Coff — Raymond C. Reese
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R. D. Conrow — William H. Rowan
W. A. Coolidge — Fred H. Ryan
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Bruce Jameyson — M. H. Walters
Bruce M. Johnson — George W. Washa
Samuel Judd — David Watstein
Lane Knight — J. C. Watt
Carl A Koerner — W. R. Waugh
James V. Mandia — Carl Weber
Henri Marcus — J. Wegrowski
Charles S. Whitney

Continued on p. 19

New Members

Of the 36 membership applications approved for the month of September there where 25 Individual, 1 Corporation, 7 Junior and 3 Student.

New York was top member producing state with 4 new member applicants. Illinois and Massachusetts shared honors for second place with 3 each.

The membership total October 1 became 4781.

Arizona

McLean, Claude E., (Indiv.) Arizona Testing Laboratories, P.O. Box 1888, Phoenix, Ariz.

Arkansas

Brummet, I. H., (Indiv.) 2007 N. Arthur, Little Rock, Ark.

California

Rhodes, A. D., (Indiv.) 3040 Lincoln Ave., Altadena, Calif.

Welsh, John J., (Indiv.) U. S.Bureau of Reclamation, Tracy, Calif.

Connecticut

Midford, Rodney W., (Jr.) Standard Builders, Inc., 55 Airport Rd., Hartford, Conn.

Florida

Scott, William E., (Indiv.) 3907 McKay Ave., Tampa, Fla.

Illinois

Germundsson, Thorbjorn, (Indiv.) 33 W. Grand Ave., Chicago 10, Ill.

Haff, John L., (Jr.) 802 E. St. Louis St., W. Frankfort, Ill.

Snowden, James R., (Indiv.) University of Illinois, 311 T. L., Urbana, Ill.

Iowa

Alsmeyer, William Carl, (Indiv.) Civil Engineering Dept., Iowa State College, Ames, Ia.

Maryland

Dietrich Bros., Inc., (Corp.) 220 E. Pleasant St., Baltimore 2, Md.

Massachusetts

Eeg-Henriksen, Kai, (St.) 125 Peterborough St., Boston 15, Mass.

Smith, Edward N., (Indiv.) 27 Worester Square, Boston 18, Mass.

Tamayo, Jose M., (St.) 34 Gorham St., Cambridge 38, Mass.

Michigan

Geer, Elihu, (Indiv.) 4001 W. McNichols Rd., Detroit 21, Mich.

Osgood, Manley, (Indiv.) Ann Arbor Construction Co., Ann Arbor, Mich.

Missouri

Mauch, Clair Allen LaVerne, (Jr.) 9310 E. 9th St., Kansas City 3, Mo.

New Hampshire

Bissonnette, Norman E., (Indiv.) 120 S. Maple St., Manchester, N. H.

New York

Kubo, Gerald G., (Indiv.) Civil Engineering Dept., New York University, New York 53, N. Y.

Manganaro, Charles Anthony, (Indiv.) 69-10A 188th St., Flushing, N. Y.

Pirozzi, Raymond, (Indiv.) 55 E. Mosholu Parkway N., Bronx, New York 67, N. Y.

Ringer, Adolph G., (Indiv.) c/o Ingersoll-Rand Co., 11 Broadway, New York 4, N. Y.

North Dakota

Fogelberg, Herbert I., (Indiv.) Garrison District Office, U. S. Army Engineers, Riverdale, N. D.

Pennsylvania

Graham, John William Jr., (Indiv.) 10419 Lindberg Ave., Pittsburgh 21, Pa.

Utah

Kemp, Clarence William, (St.) 134 S. 6th St., Tooele, Utah

Virginia

Cohen, Stanley, (Jr.) 800 Dinwiddie St., Portsmouth, Va.

Canada

Price, G. C., (Jr.) 505 Canada Bldg., Saskatoon, Sask., Canada

Cuba

Portilla, Dionisio Suarez de la, (Indiv.) 8 No. 565, Vedado, Havana, Cuba

Czechoslovakia

Cermak, Edwin, (Indiv.) Bratislava, Stefanovicova ulica 1, Vodoprojekt, Czechoslovakia

Continued on p. 19

Positions and Projects — ACI Members

Committee 323 co-sponsors prestress lecture

Committee 323, Prestressed Reinforced Concrete, co-sponsored with the University of Pennsylvania and the Philadelphia Section of A.S.C.E. a lecture by Prof. Gustave Magnel, Belgium, on prestressed concrete which was given the day of the testing of the Walnut Lane bridge prestressed girder in Philadelphia last month.

The 16-ft girder, made and tested to failure by the Preload Corp., was similar to those being used in the main span of the Walnut Lane bridge, the first prestressed girder bridge in the United States.

Directory corrections

In preparing the *1950 ACI Directory* for publication, the Institute headquarters requests each Member to check the information following his name in the *1949 ACI Directory*. Any errors in address or in the material indicating authorship, administrative committee activity, awards, etc. will be corrected in the new directory if Members will notify the Secretary's office of such changes before December 31.

ACI Members speak at PCA meeting

Six ACI Members were among the principal speakers at the 1949 regional meeting of the General Technical Committee of the Portland Cement Assn. in Los Angeles. The speakers included A. Allan Bates, PCA vice-president for research and development, Chicago; Oliver G. Bowen, consulting structural engineer, Los Angeles; Raymond E. Davis, University of California, Berkeley; F. W. Panhorst, bridge engineer, California Division of Highways, Sacramento; Bailey Tremper, materials engineer, Washington Dept. of Highways, Olympia; and Lewis H. Tuthill, manager, Field Research, U. S. Bureau of Reclamation, Denver.

More than 100 cement plant operators and officials from 12 states and Canada were present at the five-day meeting. The program included a review of PCA research activities and visits to cement plants, dams and various concrete structures in the area.

Urquhart joins consulting firm

L. C. Urquhart, Colonel, Corps of Engineers, A.U.S., retired, and ACI Member since 1923, has become associated with O. J. Porter and Co., consulting engineers, Newark, N. J. and San Francisco and Sacramento, Calif. Col. Urquhart, wartime Chief of the Engineering Division in the Office of the Chief of Engineers, has since 1946 acted in a similar capacity in the Honolulu District Office. He will be assistant chief engineer with the Porter Co. and in charge of their eastern offices. Col. Urquhart will also serve as assistant chief engineer for Edwards and Kelcey, Frederick R. Harris, Inc. and O. J. Porter and Co. in their joint office, Newark, N. J., which has been established to handle the design and construction of the highway facilities for the northern section of the New Jersey Turnpike between Linden and Route 6 near the George Washington Bridge.

Col. Urquhart, the author of several text books on civil and structural engineering, was professor and served as Head of the Structural Engineering Dept. of Cornell University prior to entering the Army in 1941.

Johnson appointed general superintendent

Harold M. Johnson, veteran of 26 years with the Hillsdale County, Mich. road commission, is now general superintendent, succeeding Fred F. Rogers, highway engineer in Hillsdale County for 25 years, who resigned to take a similar post in Livingston.

PRECAST CONSTRUCTION

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Barron and Farkas form partnership

Maurice Barron, structural engineer who recently joined ACI, and Nicholas Farkas have formed a partnership, Farkas and Barron, consulting engineers, with offices in New York City, as successors to Sigman and Farkas. Mr. Barron is a graduate of Cooper Union Institute of Technology and of Cornell University. Formerly with Clarke, Rapuano and Holleran, he was chief engineer, structural division, and developed several unusual designs of rigid frame and arch bridges including a variable skew (curved-in-plan) barrel rigid frame viaduct. He is co-author with Arthur G. Hayden of *The Rigid Frame Bridge.*

Project engineer for Pine Flat Dam

J. I. Thomas, with the Army Corps of Engineers for 18 years, has been appointed project engineer for the Pine Flat Dam and reservoir on the Kings River, east of Fresno, Calif. An engineering graduate of the University of Colorado, Mr. Thomas has assisted in the design and construction of several large multi-purpose concrete dams.

Australian building research

The Fourth Annual Report of the Building Research Laboratory of the Commonwealth Scientific and Industrial Research Organization, Australia, describes the progress of building research in Australia.

There are two organizations concerned with building research in Australia: the Commonwealth Experimental Building Station, Dept. of Works and Housing; and the Building Research Laboratory, which is a part of the Commonwealth Scientific and Industrial Research Organization. Overlapping research by the two organizations is reduced to a minimum by the operation of a Building Research and Development Advisory Council, on which the building industry is represented, and the Building Research Committee, which co-ordinates all research work.

The work of the Building Research Laboratory is divided into several sections embracing physical and mechanical testing, concrete investigations, masonry investigations, surfacing and insulation materials, acoustic investigations and organic materials.

Concrete investigations

Experimental work was reported on two main projects: an investigation of the preparation, properties and uses of foamed concrete; and a study of air entrainment. The foamed concrete report stated that resinate compounds, alkali derivatives of sulfated fatty alcohols and certain sodium alkyl aryl sulfates, stabilized with sodium carboxymethyl cellulose were found to be suitable foaming agents.

Studies were also made on the causes and means of prevention of efflorescence and deterioration of brickwork and stone masonry, masonry mortars and the efficacy of carbide waste lime in mortar.

Research is also contemplated on the use of refactory concrete for furnace linings.

We Want to Know

Continued from p. 10

Foundations

Up-to-date data would be desirable on continuous or combined foundations, raft foundations and concrete piles.

Concrete bins

Is there information on the design of circular, connected and subdivided, reinforced concrete bins where there is bending in the walls due to unbalanced loading of compartments?

Cost

Looking through past JOURNALS, there seem to be a few articles on the cost of concrete; perhaps more are needed.

Shrinkage cracks

Unreinforced concrete walls, floors-on-ground, etc., shrink progressively, building up internal stresses to the point of fracture. These fractures are uncontrolled and, while they are not structurally injurious, they are unsightly and cause unfavorable comment on the part of the public. These fractures could be anticipated and taken care of in design if it were known where to expect the cracks. Complete discussion of this subject would be appreciated.

Shell structures

I would like to see a paper on the design and construction of concrete shell structures.

Statement of the Ownership, Management, Circulation, Etc., Required by the Act of Congress of August 24, 1912, as Amended by the Acts of March 3, 1933 and July 2, 1946 of JOURNAL OF THE AMERICAN CONCRETE INSTITUTE published 10 issues a year at Detroit, Michigan for September, 1949.

STATE OF MICHIGAN... } ss
COUNTY OF WAYNE.... }

Before me, a Notary Public in and for the State and county aforesaid, personally appeared Harvey Whipple, who, having been duly sworn according to law, deposes and says that he is the Editor of the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE and that the following is, to the best of his knowledge and belief, a true statement of the ownership, management (and if a daily, weekly, semiweekly or triweekly newspaper, the circulation), etc., of the aforesaid publication for the date shown in the above caption, required by the Act of August 24, 1912, as amended by the Acts of March 3, 1933, and July 2, 1946 (section 537, Postal Laws and Regulations), printed on the reverse of this form, to wit:

1. That the names and addresses of the publisher, editor, managing editor, and business managers are:

Publisher, American Concrete Institute, 717 New Center Bldg., Detroit 2, Mich.

Editor, Harvey Whipple, 717 New Center Bldg., Detroit 2, Mich.

Managing Editor, None.

Business Manager, None.

2. That the owner is: (If owned by a corporation, its name and address must be stated and also immediately thereunder the names and addresses of stockholders owning or holding one percent or more of total amount of stock. If not owned by a corporation, the names and addresses of the individual owners must be given. If owned by a firm, company, or other unincorporated concern, its name and address as well as those of each individual member, must be given.)

American Concrete Institute, 717 New Center Bldg., Detroit 2, Mich.

Herbert J. Gilkey, President, Iowa State College, Ames, Ia.

Frank H. Jackson, Vice-President, U. S. Public Roads Administration, Washington, D. C.

Harry F. Thomson, Vice-President, Material Service Corporation, Chicago 2, Ill.

3. That the known bondholders, mortgagees, and other security holders owning or holding 1 percent or more of total amount of bonds, mortgages, or other securities are: (If there are none, so state.)

NONE.

4. That the two paragraphs next above, giving the names of the owners, stockholders, and security holders, if any, contain not only the list of stockholders and security holders as they appear upon the books of the company but also, in cases where the stockholder or security holder appears upon the books of the company as trustee or in any other fiduciary relation, the name of the person or corporation for whom such trustee is acting, is given; also that the said two paragraphs contain statements embracing affiant's full knowledge and belief as to the circumstances and conditions under which stockholders and security holders who do not appear upon the books of the company as trustees, hold stock and securities in a capacity other than that of a bona fide owner; and this affiant has no reason to believe that any other person, association, or corporation has any interest direct or indirect in the said stock, bonds, or other securities than as so stated by him.

5. That the average number of copies of each issue of this publication sold or distributed through the mails or otherwise, to paid subscribers during the twelve months preceding the date shown above is.....(This information is required from daily, weekly, semiweekly, and triweekly newspapers only.)

HARVEY WHIPPLE,
(Signature of editor)

Sworn to and subscribed before me this 19th day of September, 1949.

JEAN M. STEM, Notary Public
(My commission expires Oct. 17, 1952)

[SEAL]

Honor Roll

Continued from p. 14

H. F. Williams
Paul C. Winters
M. O. Withey
Thomas Worcester
K. W. Wright
Charles E. Wuerpel
Alexander H. Yeates
R. B. Young
Robert Zaborowski

New Members

Continued from p. 15

Ecuador

Smith, Harold T., (Indiv.) P.O. Box 630, Quito, Ecuador

England

Smith, Anthony d'Oyly, (Indiv.) 9 Rugby Court, Richmond, Surry, England

India

Nayar, D. P., (Indiv.) 20, Patel Park, Amballa Cantonment, East Punjab, India

Norway

Giaever, Arne, (Indiv.) Norsk Hydro, Solligt. 7, Oslo, Norway

Panama

Parada, Ramiro, (Jr.) Box 374, Panama, R. P.

Turkey

Taskin, Ferruh, (Jr.) Betonarme Kursusu asistani, Istanbul Teknik Universitesi, Istanbul, Turkey

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SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 21 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES (ACI 616-49) 46-1

Price 50 cents (in special covers).

REPORT of COMMITTEE 616—Sept. 1949, pp. 1-16 (V. 46)

Supersedes 38-30 and 45-18.

This ACI standard establishes recommended practices for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

AN ULTRASONIC METHOD OF STUDYING DETERIORATION AND CRACKING IN CONCRETE STRUCTURES 46-2

Price 35 cents.

J. R. LESLIE and W. J. CHEESMAN—Sept. 1949, pp. 17-36 (V. 46)

A new method and apparatus for field and laboratory testing of concrete is described. The apparatus called the "Soniscope" was originally designed to detect internal cracks in concrete. It develops pulses of ultrasonic sound in the material and measures the velocity of their transmission through it. This pulse velocity has the unique advantage of being independent of the size or shape of the body under test. Measurements can be made with equal facility in mass concrete, slabs or laboratory specimens.

The existence and extent of internal cracks and the depth of visible surface cracks can be determined by the use of this apparatus.

The velocity has been found, by experiment, to be a reliable measure of the condition of the concrete and is particularly useful in deterioration studies. The dynamic modulus can be calculated from this velocity, and values so obtained are found to agree closely with the results of tests using established methods.

MANUFACTURE OF REINFORCED FOAM CONCRETE ROOF SLABS ... 46-3

Price 35 cents.

I. T. KOUDRIASHOFF—Sept. 1949, pp. 37-48 (V. 46)

The Russian type of lightweight concrete described used a rosin-glue emulsion to preserve the air voids before the initial set of the cement. Shrinkage was decreased and strength increased through high pressure steam curing. The autoclave treated foam concrete used in the production of precast industrial roof slabs had a unit weight of 47 lb per cu ft and a compressive strength of over 500 psi. The lightweight slabs, used in a load carrying capacity and as insulation, reduced construction time by 50 percent and costs by as much as 20 percent. Test data on roof slabs and production procedures are also described.

SUGGESTIONS ON CONCRETE FLOOR CONSTRUCTION 46-4

Price 35 cents.

ERNST GRUENWALD—Sept. 1949, pp. 49-56 (V. 46)

The relationship between good concrete floors and the proper selection of cement and aggregates is discussed. Data are cited to emphasize the advantage of coarse-aggregate mixes over cement-sand topping for concrete floors.

USE OF AIR-ENTRAINING CONCRETE IN CANAL LINING 46-5

Price 35 cents.

JOSEPH J. WADDELL—Sept. 1949, pp. 57-64 (V. 46)

This paper presents a discussion based on field observations of the use of air entrainment in canal lining concrete. The Bureau of Reclamation made studies of air-entraining agents for use in the irrigation canal lining for the Friant-Kern Canal on the Bureau's Central Valley Project in California. Results to date indicate that appreciable benefits accrue when an air-entraining agent is used in concrete which is placed and compacted by means of a mechanical slip-form. Care is necessary in adjusting concrete mixes to incorporate entrained air because of the sensitivity to mix changes of concrete for slip-form placement.

THE USE OF PORTLAND-POZZOLAN CEMENT BY THE BUREAU OF RECLAMATION .. 46-6

Price 35 cents.

ROBERT F. BLANKS—Oct. 1949, pp. 89-108 (V. 46)

The Bureau of Reclamation has made extensive studies of pozzolanic materials, and portland-pozzolan cements are now being used in many of the major structures built by the Bureau. The properties of portland-pozzolan cement that are used advantageously in the production of mass concrete are described.

RESISTANCE OF CONCRETE AND PROTECTIVE COATINGS TO FORCES OF CAVITATION 46-7

Price 35 cents.

WALTER H. PRICE and GEORGE B. WALLACE—Oct. 1949, pp. 109-120 (V. 46)

A machine used for producing cavitation erosion in the laboratory is described and the results of tests made to investigate the effect of mix proportions, surface treatment, and protective coatings on the resistance of concrete to cavitation are discussed. Through proper use of these, the resistance of concrete surfaces to cavitation erosion may be extended three or four times, but even the best concrete will not resist the forces of cavitation for a prolonged period. Heavy rubber coatings bonded well to the surface of the concrete have proved effective.

VACUUM PROCESSES APPLIED TO PRECAST CONCRETE HOUSES ... 46-8

Price 35 cents.

K. P. BILLNER and BERT M. THORUD—Oct. 1949, pp. 121-128 (V. 46)

The use of vacuum processes in precast concrete construction simplifies the building of fire-resistant, durable structures designed to permit the maximum use of like units. Vacuum processes have thus far been used in one- and two-story houses and one-story industrial or warehouse structures, but further developments should make possible similar benefits for multi-story construction.

The specific vacuum methods utilized are: (1) extracting excess water from freshly placed concrete prior to set, thereby increasing early strength and enabling early handling of units, (2) holding forms in place by vacuum and (3) handling and placing finished and hardened concrete units by cast-in-place closures formed and quickly hardened by vacuum processes.

The use of these methods results in high strength monolithic construction having exterior and interior finished surfaces, with insulation incorporated into the construction, if desired.

NEW TYPE OF CONSISTENCY METER TESTED AT ALLATOONA DAM.....46-9

Price 35 cents.

JAMES M. POLATTY—Oct. 1949, pp. 129-136 (V. 46)

Various mechanical devices have been developed for indicating consistency of concrete since the early days of visual inspeciton after discharge from the mixer. A new type meter to measure the consistency of concrete while it is being mixed was tested at Allatoona Dam and the operation and results are described.

EFFECT OF MIXING SEQUENCE ON THE PROPERTIES OF CONCRETE....46-10

Price 35 cents.

F. L. FITZPATRICK and W. SERKIN—Oct. 1949, pp. 137-140 (V. 46)

The order in which the constituents of concrete (aggregates, cement and water) are combined in the mixing operation, has a significant effect upon the properties of the concrete as to workability, strength, density, surface finish and absorption. Tests are reported.

BOND OF CONCRETE REINFORCING BARS.....46-11

Price 35 cents.

ARTHUR P. CLARK—Nov. 1949, pp. 161-184 (V. 46)

The tests reported were made to compare the resistance to slip in concrete (bond) of deformed bars when tested in beams and companion pull-out specimens, to secure information on the effects of size of bar, the type of deformations on the bars and the strength of concrete on the bond. The bars were cast in a horizontal position in all test specimens. The variables were depth of concrete under the bar, length of embedment of the bar in the concrete, strength of concrete and diameter of bar. Slip of the bar was measured at the loaded and free ends. Three tests were made with 2 in. of concrete under the bar, with 15 in. of concrete under the bar and with 3-in., 12-in., and 16-in. embedments.

Bond strengths for the beams and the pull-out specimens were affected similarly by changes in the geometry of the bars and the bond test specimens. They were greater when the bars were near the bottom than when they were near the top of the specimens. The highest bond strengths were obtained with bars having deformations conforming to suggested requirements for maximum spacing and minimum height and providing ratios of shearing to bearing areas less than 10, usually less than 6.

PERLITE AGGREGATE: ITS PROPERTIES AND USES.....46-12

Price 35 cents.

J. JOHN BROUK—Nov. 1949, pp. 185-192 (V. 46)

Synthetic expanded volcanic rock, better known as perlite is a fairly recent addition to the lightweight aggregate field. Its use in concrete is governed by weight, gradation, mixing procedure, cellular structure, strength of cell walls, insulating properties, cost, etc. Air entrainment appears to be necessary to give a workable, nonsegregating mix. Perlite aggregate blended with sand can be used in high strength structural concrete and concrete products.

THIN WALL CONCRETE SHIP CONSTRUCTION.....46-13

Price 35 cents.

FRANCIS R. MAC LEAY—Nov. 1949, pp. 193-204 (V. 46)

The development of construction methods for placing thin concrete walls is described. After several attempts using the standard method of pouring between double forms, a new method was developed which permitted the successful construction of 5/8-in. and 1 1/2-in. concrete walls. Tests were also conducted to determine the practicability as well as the strength of Gunite as a medium to unite precast units into a monolithic ship. Later these methods were used successfully in the construction of a concrete landing craft for the United States Navy.

SPECIFICATIONS SHOULD BE REALISTIC.....46-14

Price 35 cents.

HARRY F. THOMSON—Nov. 1949, pp. 205-220 (V. 46)

Specifications for concrete in moderate-sized and lesser construction frequently contain provisions which are ambiguous, conflicting in application, or do not fully recognize local materials. Most of these questionable features result from (1) inadequate information regarding the characteristics of concrete, or (2) use of ready-written specifications without revision for conditions or changes in standard requirements.

Among the features discussed are: specifying both method and result; "frozen" specifications; habitual use of "1-2-4"; multiple provisions for quality; strength without naming consistency; recognition of local materials; use of compression tests; time of placing; bin-test of cement. Numerous quotations from specifications are given, and suggestions offered for bringing the provisions in line with actual conditions.

46TH ANNUAL ACI CONVENTION

February 20-22, 1950

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DISCUSSION

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Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)—Report of Committee 616

An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures—J. R. Leslie and W. J. Cheesman

Manufacture of Reinforced Foam Concrete Roof Slabs—I. T. Koudriashoff

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Thin Wall Concrete Ship Construction—Francis R. Mac Leay

Specifications Should Be Realistic—Harry F. Thomson

Inspection of cement, aggregates and other materials at the manufacturer's plant before shipping to the job-site helps in getting better materials for better concrete.

Inspection and Testing of Materials*

By NICOLAAS T. F. STADTFELD†

SYNOPSIS

This paper sets forth the care taken in the inspection and testing of 7 million barrels of portland cement used in 4 million cu yd of concrete of great uniformity and every indication of durability after 12 years of observation. It stresses for the concrete the importance of low water solubility, freedom from "laitance," lack of cement burns of the workers, low alkali content, and it shows above all the necessity for manufacturing control of the cement clinker. It describes how premature set of concrete was prevented. It also deals with the inspection, grading, and testing of aggregates. The Board of Water Supply specifications for aggregates are given as well as other factors used in securing good material.

INSPECTION OF CEMENT

Inspection and testing of materials is a broad field, justifiable only if the end product, resulting from inspection and testing of the component parts, is suitable for the intended purpose and of such durability as can be reasonably expected under the attack by nature's forces.

The quality of concrete, the end product, is naturally dependent on the quality of the cement, aggregates, water, and in many cases, reinforcing steel.

Contrary to the practice of other investigators, the engineers of the Board of Water Supply, after studying concrete failures in their own work, concentrated their research on portland cement, while admitting that it might not be the only cause of disintegration. This research since 1912 culminated in the specifications written by the late Thaddeus Merriman for the Board of Water Supply, City of New York, and applied since 1937.

The nearly 4 million cu yd of concrete made with this cement are uniformly of excellent quality, and this one fact is considered of enough

*Received by the Institute Oct. 10, 1949. Scheduled to be presented at the ACI 46th annual convention, Chicago, Ill., February 20-22, 1950. Title No. 46-15 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 4, Dec. 1949, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Apr. 1, 1950. Address 7400 Second Boulevard, Detroit 2, Mich.

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importance to justify the writing of this article, which is simple and can be read some morning at breakfast instead of hiding behind a newspaper. There will be found no intricate nor brilliant research explaining failures, which still fill many pages of concrete literature, because there has been none.

Our inspection differs from the usual way of accepting a finished product in that we want to know what goes on in the kiln room. In this respect we do not discriminate against cement companies as we are just as inquisitive about other materials, having representatives present during forging and during the pouring of large or small castings whether they are made of bronze, steel, stainless steel, or cast iron. It is only by following through a complete process that we have been able to make progress.

In the case of cement clinker this procedure is considered of utmost importance. Where metals are concerned we at least get to deal with a liquefying process tending towards an even distribution of constituents and, in addition, in castings the impurities are brought to the top as part of the risers and discarded.

A cement clinker, however, as commercially produced never liquefies but is subjected only to incipient fusion, which means that conditions must be carefully controlled to change the molecules in the raw mix to entirely different ones in the clinker. How can this be done unless kiln temperatures and kiln rotations are recorded? While there are many other factors influencing the end product, the two mentioned are so simple to check on that they are incorporated as necessities in the specifications and their recording is one of the duties of the inspector. Whenever trouble is encountered, unsatisfactory clinker is by-passed. An additional check on the burning process is provided by the Merriman sugar test, which is now so well known that it will not be described here.

INSPECTOR'S DUTIES AT CEMENT PLANT

Some of the inspector's duties at the cement plant during burning, grinding, and shipping are:

Making and sampling cement

1. Bin to be filled must be swept clean if other than Board of Water Supply cement was used on previous filling.
2. All lines, conveyors, elevators and tube mills must be cleared of other cements before grinding BWS cement.
3. A sample of cement must be obtained for every 400 barrels ground.
4. Check sampler to see that adequate quantity is obtained to fill a gallon can, plus the quantity needed for the mill sample during the 400-barrel run.
5. The cement taken from the sampler must be thoroughly mixed before filling the sample cans (either by rolling on oil cloth or mechanical means).

6. If clinker is stored, make sure no rain water can get to the clinker pile through leaks in the roof or side openings.

7. Approximate a 400-barrel run by means of clinker scales, mill production, depth measurements, or any other means available at the mill.

8. Two samples of clinker must be taken for each grind, one before cooling water is added or before going to the cooler, and one after the cooler or before it goes into the grinding mill without the gypsum added.

9. A complete history of the clinker must be put on a card and put into the can with the clinker sample.

10. Covers of all sample cans, both of cement and clinker, must be soldered in two or three places before shipment to the laboratory.

11. A complete record of both temperatures and revolutions of all kilns must be recorded on the sheets provided for the purpose.

12. Indicate on these sheets any breakdowns, red spots, or any other defects in burning that would show why temperatures dropped or speeds changed.

13. Keep a log of all samples taken, showing the bin number, time taken, date taken, and number of tube mills operating.

14. After bin or silo is completely filled, seal all openings on the top and the bottom. Bottom seals to be broken when shipment is made from the bin. Top seals are only to be broken when emptying a bin and when grinding into a bin, but at no other time.

Shipping cement

1. Check to see if accepted bin is being used for loading.

2. All lines, conveyors, elevators and loading bins must be cleared of other cements before starting to load BWS cement.

3a. See that all bulk cars are clean before loading is started.

b. Cement being loaded in bulk cars must be fully protected from rain or snow.

4. If bags are being loaded, see that the BWS stamp having the correct month and day is being used. Every bag must be stamped.

5. Record the weights of enough bags from each packing machine to insure proper weight for each car shipment and show the net average on weight cards which are to be sent to the home office only.

6. Check car numbers, contract numbers, contractors and destination with shipping clerk at the mill.

7. Put BWS seals on all car openings, top, bottom and sides, after loading is completed.

8. Indicate on the shipping cards when a bin is emptied.

These are the hard and fast rules, but their successful application is still dependent on the intelligence and personality of the inspector. Inspectors are chosen with great care and instructed to realize that we are not policemen swinging clubs but present at the plant in a spirit of cooperation. Long contact with chemists and other personnel of cement plants has brought the conviction that they are the salt of the earth, anxious to provide the consumer with the product his specifications call for. One of the functions of the inspector, therefore, is to prevent honest mistakes rather than any deliberate negligence. This cooperation has been a great factor in ironing out difficulties which still may arise in spite of the best of efforts.

Reference is made to the phenomenon of premature set in concrete which may cause it to harden before it gets into the forms. This occurred in one tunnel job and was most embarrassing. A few days were spent at the cement plant and the difficulty settled once and for all. The clinker was allowed to cool to a lower temperature before grinding, the grinding rate stepped down to lower the heat development during grinding and, in addition, the gypsum content lowered. Silos already accepted with cement showing an SO_3 content of 1.4 percent or over were withdrawn by the cement company and our job proceeded without further difficulties. This solution may not be the proper one in every case, but it certainly works in the case of a hard-burned clinker of low alkali content of a composition paralleling Type II cement.

One wonders why other consumers keep on trying to place concrete with premature set, paying the contractor enormous sums for the extra work in placing it in the forms, or being sued for such sums, when a visit to the cement plant, a discussion with its chemist, and an abiding by his superior knowledge, might straighten out the difficulty at the start.

SPECIAL ADVANTAGES

The investment made by the BWS in a somewhat more costly inspection has been considered as one of the best ever made. Seven million barrels of cement have shown the greatest uniformity ever attained, as our rather elaborate records indicate. A few special advantages are:

1. Since we are dealing with water supply it is important that concrete exposed in tunnels to (in our case) soft waters with fairly high velocities, show great resistance to water solubility. A hard-burned clinker low in alkali, ground with a minimum amount of gypsum, results in a cement many times less soluble than other cements investigated. (See Fig. 1a and 1b)
2. This cement, practically free from underburned particles, will not produce "laitance" to any extent, even in 20-ft lifts of concrete, thus adding to durability. (See Fig. 2a and 2b)
3. The distress and loss of time due to "cement burn" of the men coming in contact with concrete during its placing in shafts and tunnels has been practically eliminated, mostly due to the low alkali content.
4. The properly burned cement, low in alkalies, provides a period of grace during the placing of concrete by modern methods. There are no flash sets of underburned particles in the mass, nor undue temperature rises due to high alkalinity, even when the concrete is disturbed after two hours, due to breaking down of equipment. This is considered as one of the outstanding characteristics.

It seems a fair question to ask if others who do not follow through as we do would obtain the same results and, if so, why bother? The negative answer is predicated on long experience. It stands to reason that, when a kiln is started after a shutdown, the first clinker produced is decidedly underburned as the kiln temperature has to be raised gradually. Also, frequent breakdowns occur, and it would amaze many a consumer to

Fig. 1a—Side view

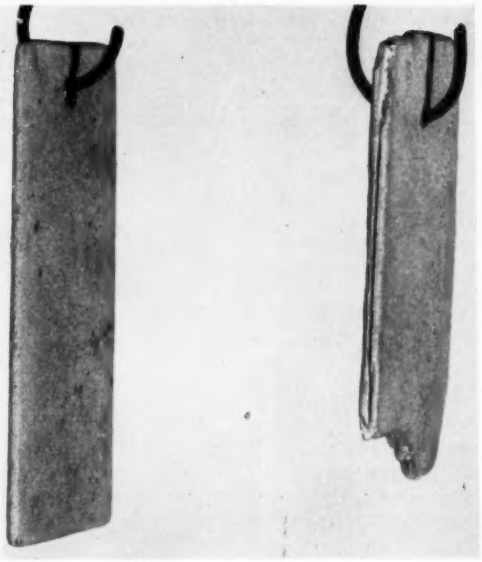

Fig. 1a and 1b indicate clearly the difference in water solubility between a 2 x 4-in. neat cement slab made of Merriman cement, exposed to distilled water, which was renewed on the average of twice a week, and a slab of one of the better Type I cements, exposed for the same length of time. Note the cracking of the latter in the side view and almost total disintegration in the front view. In general, this Type I cement fails in 6 months while the Merriman cement is still good after 2 years.

This test was developed by N. Goodman, Senior Chemist, Board of Water Supply.

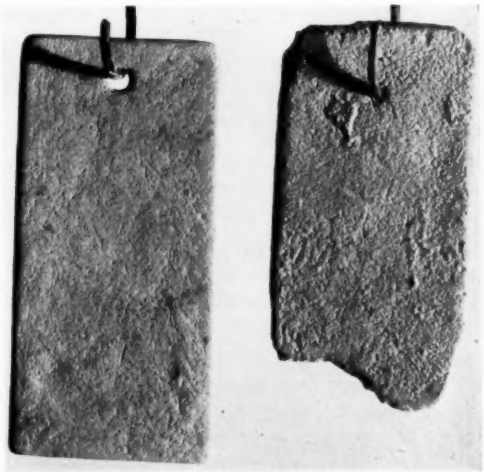

Fig. 1b—Front view

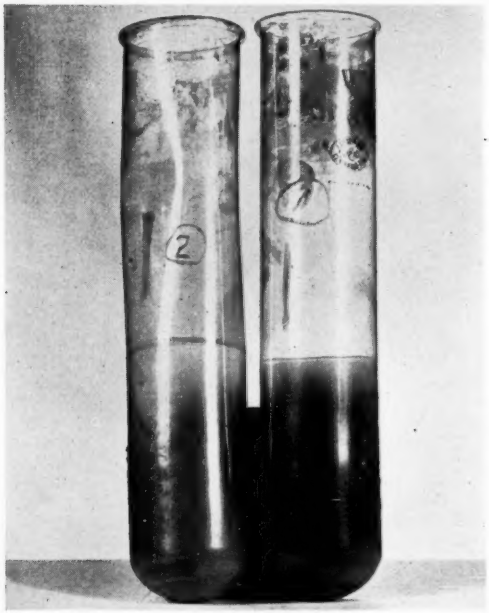

Fig. 2a—Specimens still in the plastic centrifuge tube

Fig. 2a and 2b indicate clearly the difference in the amount of laitance, shown as light colored substance, between a centrifugally spun sample of Merriman cement (1) and one of the better Type I cements (2).

Except for a higher SO_3 content in the laitance there is not much difference in chemical analysis between it and the rest of the specimen. Its weathering resistance however is extremely poor. It consists of underburned particles.

This test was developed and the specimens supplied by R. R. Proctor, Field Engineer, Department of Water and Power, Los Angeles, Calif.

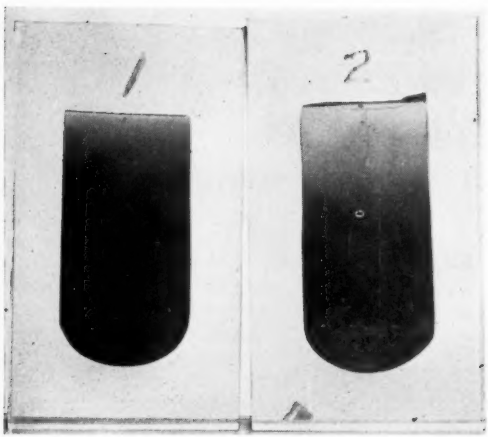

Fig. 2b—Specimens with the tube removed, cut in half, and preserved in plastic

note how much clinker is by-passed during the making of Board of Water Supply clinker. Add to this that, to my knowledge, no clinker is ever discarded, nor is it ever given a second burning. Many tons of clinker, exposed for many years to the weather after the closing down of one cement plant, were bought up cheaply a few years ago by another one, and mixed with its clinker. All I can say is: Poor consumer!

These statements are made because portland cement seems to many to be sacrosanct and, when failures occur, quite often there seems to be a tendency to start out with a clean bill of health for the cement used.

I refer to the excellent article by Ruth Terzaghi appearing in the ACI JOURNAL, June 1948, entitled "Concrete Deterioration in a Shipway," which deals with the distress in the north pier forming the lateral support for floating gates constructed in 1940 and 1941 at Newport News, Va. When this article was presented at the 1947 meeting in Denver, Colo., the type of cement used in this pier and in the middle and south piers which showed no distress was not mentioned. Only a request from the floor brought out the facts now mentioned in the article that there was a difference; namely, Type I cement used in the unsatisfactory pier, and Type II in the other two. The question arises why should portland cement be considered so sacrosanct. It is neither fair to the consumer, who should be informed about the actual facts, nor to the producer of the Type II cement who apparently did an excellent job.

AGGREGATE SPECIFICATIONS

Our specifications for concrete aggregates are:

Fine aggregate for concrete and mortar shall be natural sand of such acceptable size and quality as are approved. All sand shall consist of clean, hard, strong, durable, insoluble and uncoated particles. It shall contain not more than one percent by weight of vegetable matter, nor more than three percent by weight of clay or silt, and shall be free from such quantities of other deleterious substances as will render it unsuitable. All sand shall be made clean by thorough washing. Any fine aggregate which breaks down into unacceptably fine material on boiling in water, or ten percent or more of the dry weight of which is soluble in a 1 to 1 solution of hydrochloric acid in 15 minutes, shall be rejected. Fine aggregate shall contain both fine and coarse particles, with none larger than will pass a ¼-in. mesh screen, and with such gradation in size of particles as will produce smoothly working concrete without tendency for the water to drain away. In general, sands containing an excess of fine grain sizes will not be acceptable. Sands shall be subject to tests for organic impurities by the colorimetric method, using a 3 percent solution of sodium hydroxide; also to tension mortar tests, and shall show strengths not less than those of similar tests made with standard Ottawa sand.

Coarse aggregate for concrete shall be approved gravel or fragments of acceptable stone broken to such sizes as are herein specified. A mixture of broken stone and gravel may be used, but only as permitted. Coarse aggregate shall consist of clean, hard, strong, durable, insoluble and uncoated pieces and shall not contain undesirable quantities of such that are soft, friable, thin, elongated or laminated; it shall be free from

such dust, alkali, organic material, loam, clay, mica, schist or other deleterious substances as will render it unsuitable, and it shall be thoroughly washed so as to be clean at the time it is mixed in concrete. Pieces of coarse aggregate, approximately 1-in. cubes, shall, when immersed in 600 cc of a 1 to 1 solution of hydrochloric acid, show a loss from their original dry weight of not more than ten percent in 45 minutes. Coarse aggregate shall be of the sizes suited to the various parts of the work and shall be kept separate according to size before being used in the concrete. The coarse aggregate shall be stored in the following sizes: (1) passing a 1½-in. mesh sieve and retained on a ¾-in. mesh sieve and (2) passing a ¾-in. sieve and retained on a ¼-in. mesh sieve.

Each size of coarse aggregate shall be graded from fine to coarse within the limits shown, to the satisfaction of the engineer. A greater maximum size for coarse aggregate may, however, be required in some parts of the work, while in others the maximum size may not be permitted to exceed ¾ in. Coarse aggregate shall be kept in separate hoppers from fine aggregate, and any mixture of fine and coarse aggregate delivered to the work shall be screened and remixed in ordered proportions before use.

Samples of aggregates which the Contractor proposes to use shall be submitted to the Engineer at least two weeks before the Contractor plans to commence delivery at the site of the work. Aggregates shall not be delivered until the samples shall have been approved and, as delivered, they shall be in all respects equal to the approved samples. Samples of fine aggregate (about one quart) shall be submitted in glass jars with stoppers, and samples of not less than one cu ft of coarse aggregates in suitable boxes or bags. All samples shall be plainly and neatly labeled with the place from which taken, where proposed to be used, the date, and name of collector.

It will be noted that requirements for sodium sulfate and magnesium sulfate resistance are missing. This is because we have never been able to tie in such resistance or the lack of it with performance of the aggregates in concrete, so why mess with such messy tests. On the other hand our main bugaboo in the past has been the presence of unweathered shale in the sand which contributed to poor weathering qualities of the concrete. For this reason the boiling test is incorporated in the fine aggregate specification.

Since we are confronted in our work with a flow of soft waters through concrete tunnels, water solubility is again a factor. Siliceous aggregates are therefore preferred and carbonate aggregates such as lime carbonate and dolomite, are considered less satisfactory. The acid test in both fine and coarse aggregates is a safeguard in this respect.

It may be of interest to point out that the specifications are broadly written with the end product, a smoothly working concrete, emphasized and the grading left to the judgment of the engineer. In each case the producer is supplied in advance with the required limits which in general are as given in Tables 1 and 2.

The 3 percent upper limit passing the 100 mesh sieve may be somewhat startling to those who in later years, have allowed a considerably higher percentage. It is based on the experience that while some of these finer materials may be beneficial, others are devilish, coating the sand particles and preventing bond or going into mud. It is difficult to

TABLE 1—FINE AGGREGATE GRADING

Sieve size or number	Cumulative percentage passing
¼ in.	100
4	95–100
8	80–90
16	40–70
30	20–50
50	12–30
100	0–3

TABLE 2—COARSE AGGREGATE GRADING

Sieve size, in.	¾ in.-1½ in. Aggregate, percent	¼ in.-¾ in. Aggregate, percent
1½	95–100	
1	30–70	
¾	0–8	95–100
½		40–80
¼		0–7

predict how they will behave and considered safer to keep them at a minimum. The tolerances on the ¾-in. and ¼-in. sieves are necessary for plant operation as no manufacturer can work any closer when producing on a large scale.

AGGREGATE INSPECTION

So much for grading, which is looked upon as a guiding factor, necessary but somewhat theoretical, and never allowed to interfere with the judgment of a good concrete inspector, who is the final judge of what constitutes a smoothly working concrete.

Since the furor caused by aggregate-alkali reaction, we check up on such possibility in virgin deposits. However our cement since 1937 has contained less than 0.5 percent of alkali computed as Na_2O, so the chances of disintegration due to this cause are small. If your mind has not wandered until you read this far, just think of the many excellent articles that would not have been written and the many important and less important structures that would not now be subject to disintegration if Merriman cement had been universally adopted at that time.

From a practical point of view, inspection of aggregates of necessity must be made at the source. No barge load of gravel can ever be sampled for grading after it arrives in the harbor, so inspectors are present during loading and processing, taking frequent samples from the loading belt even though this may necessitate their presence on a dredge in Long Island Sound during the wee hours of the morning.

The most important aid in making a decision on suitability of aggregates is their service record in concrete constructed in years gone by. If this is not available, the services of our geologist are called upon and new deposits are extensively examined either by the aid of numerous test pits or by a high steam shovel cut.

In the case of Long Island aggregates, which have a long, excellent service record the accent is mainly on cleanliness and grading. For new deposits, much time is spent on preliminary work, which is considered a good investment. The contractor decides on the property he wishes to lease or buy, then digs as many as 60 test pits 20 ft deep, well braced. As excavation proceeds a small pile of material representing each five ft excavated, is set aside on top, clearly marked for depth. After this is done, many days may be spent walking over the deposit with the contractor's representative and deciding on the part of the deposit which should be used. Some of it may be worthless fine silt or clay, some may be lacking in fine aggregate. It may well be said that it is the contractor's business to provide suitable aggregate but practice indicates that many difficulties in later processing may be thus prevented.

Our main difficulties arise from the presence of shale, and deposits containing shale are carefully avoided. Where a sandstone gravel is encountered, thin sections are made to determine whether the cementing material between the grains is of a siliceous or argillaceous nature, the latter being considered' unsatisfactory.

Where suitable rock is excavated from the tunnels this is often used for coarse aggregate. Again the geologist comes to the front and breaks in field men to supervise the discarding of unsuitable veins such as feldspars, pyrites and shale.

As a final check concrete cylinders are made to determine the compressive strength of the concrete, since in spite of all precautions taken there may still be unknown factors at work which might be detected by this method. There are no requirements for their strength, all our concrete designs being based on cement factor and the necessity of a smoothly working concrete.

OTHER MATERIALS

Reinforcement

This will read like an anti-climax. Reinforcing bars with approved deformations are accepted on certified mill test reports, showing results

of chemical composition and physical tests. Through many, many years of experience and many check-ups we are convinced that these reports truly represent the quality of the material. In cases where rods have to be taken out of stock of unknown origin, as may occur in an emergency, a check-up is, of course, made by the laboratory.

Water

Potable waters are considered satisfactory for concrete construction.

RECAPITULATION

A description has been given of the methods of inspection and the reasons therefor. While the requirements of inspection at the source, meaning the burning of the clinker, storage, and grinding, may seem a hardship on the manufacturer, it is believed that the results obtained warrant this. On the credit side it should be stated that so much faith is placed in this manufacturing control that frequently silos are accepted on chemical analysis and the one day sugar test only, strength tests becoming of minor importance as their requirements are invariably met.

Discussion of a paper by Nicolaas T. F. Stadtfeld:

Inspection and Testing of Materials*

By ROBERT F. BLANKS, HENRY COMACK, ALDEN M. KLEIN and AUTHOR

By ROBERT F. BLANKS†

The production of uniform concrete is essentially a manufacturing operation, and like a good many other manufactured materials, there are a lot of ingredients that go into it. Basic to the production of the final, uniform product of concrete is the production of uniform ingredient materials that go into it.

Now let's see where we have progressed in the manufacture of those ingredient materials in a uniform manner, so that in the end they can be combined in the concrete to produce uniform concrete.

Looking at the production of aggregate, for example, in the past few years the development of adequate and efficient equipment to produce uniform aggregates has taken long strides. We have taken a good bit of the variable human element out of the production of uniform aggregates, and by some measure of control on how those production processes are carried out, we can be reasonably sure that into each batch of our concrete goes a reasonably uniform bit of this aggregate and that aggregate and the other aggregate.

Now let's take a look at the methods by which those materials are proportioned out to go into the concrete mix. Again, the developments in modern batching equipment are little short of magic. As a matter of fact, some of our modern batching and mixing plants have been called veritable houses of magic because of the automatic way in which they accomplish the job they have to do, and, again, have taken much of the variable human element out of the batching process.

Let's come on down to the mixing. We don't find the comparable progress in mixing equipment that we find in the processing and batching equipment. True enough, the equipment does a reasonable job of putting the ingredient materials together, coming out with uniform blending and so on, but it does

*ACI JOURNAL, Dec. 1949, *Proc.* V. 46, p. 237. Disc. 46-15 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

†Chief of Research and Geology Division, U. S. Bureau of Reclamation, Denver, Colo.

not do it in an efficient way but simply through brute force, if you want to put it that way.

Then let us consider the other important ingredient in concrete—cement. In some of the larger organizations we have gone just about as far as we can go in controlling all the various operations on the job to insure that we have a concrete product which, day to day, week to week and month to month, will be as nearly uniform as it is practical to make it. However, unless we have uniform ingredients to go in there in the first place, it is still going to be variable in the end.

We will find, for example, that a cement from a given plant will vary all over the map from week to week, far more than it should, particularly as to water requirement, setting characteristics and strength development. We find that cements manufactured under the same specifications in different plants will vary still further. And if we go into the production of cement —which is a manufacturing process in itself, because a lot of ingredient materials are dug out of the ground and put together to form the final product —unless the raw materials are uniform, the cement which is made from them is not going to be uniform.

In our control of concrete work, the cement is probably the least uniformly controlled of any material or operation; and the writer throws that out as a challenge to our cement friends.

By HENRY COMACK*

The writer is in total agreement with the spirit of Stadtfeld's paper. It is fundamental that inspection and testing of materials is justifiable only if the end product is suitable for the intended purpose. At a concrete batching plant, a function of the inspector is to cooperate with and represent the resident engineer in all matters of concrete control. In a large city, a great deal of controlled concrete originates from ready-mixed plants. Accordingly, there may be stationed in one plant inspectors representing various entities such as city departments, counties, state and federal departments, consulting engineers, architects, testing laboratories, bi-state commissions, etc. Under such conditions, the writer agrees with the author, that successful inspection depends on the inspector's intelligence and personality. His all-important function is to enforce the specifications assigned by his firm or agency. But, owing to the unique conditions found in ready-mixed plants, it is suggested that the following should be added to his responsibilities:

1. Get along with and be agreeable in the presence of other inspectors.
2. Do not monopolize the testing equipment; take care of the equipment found at the plant and clean up after finished.
3. Function as an individual as far as official duties are concerned. Keep all complaints made to the plant representative confidential.
4. Do not interfere with the plant personnel or operation. Do not abuse facilities such as telephone, etc.

*Civil Engineer, Flushing, N. Y.

5. Do not pass on information regarding business activities of a ready-mixed plant.

By the same token, owners of ready-mixed plants should realize that inspectors are "here to stay," and that they too have responsibilities. As observed by the writer, facilities for inspectors are sadly neglected in many instances. The following suggestions are made to ready-mixed concrete plant owners:

1. A separate room, preferably on the batch floor, provided with tables, desks, chairs, drawers for records and facilities for tests.
2. Provide lockers and washing facilities at least equal to those provided for plant employees.
3. Convenient access to a telephone.
4. Provide safe stairs, passageways, protection from injury from falling aggregates, moving equipment, slippery walks, etc.
5. Car-parking facilities located on or near the plant; such facilities should provide protection from theft, collision, cement dust and debris.

The writer wishes to point out that many ready-mixed companies have recognized the value of good relations with inspectors and provide facilities as suggested above.

It is also desired to discuss the desirable background of a good concrete plant inspector. The writer is of the opinion that better concrete can be obtained if more *practical* men participate in inspection, testing and research. Unfortunately, we find a great deal of testing and research done by men who have graduated directly from college into the testing laboratory. Based upon contact with the men that handle and place concrete, the writer believes that the theoretical and practical concepts of concrete would be enriched if practical know-how were incorporated with testing, inspection and research. Examinations of old masonry specimens indicate the "feeling" and capability for handling concrete by practical men. A combination of education and practical experience appears to be the answer. The practice of sending recent graduates to inspect concrete is less deplorable than the use of inspectors with neither education or experience. In many cases, honesty, integrity, intelligence and hard work make up for the lack of background; but we should strive to have men with the virtues mentioned *plus* experience and education.

By ALDEN M. KLEIN*

The paper under discussion indicates with considerable clarity the advantages of testing cement and aggregates prior to their incorporation in concrete.

It is also implied that there are variations in the characteristics of different lots of cement, even when shipped from the same mill. These variations are well known, at least by the cement manufacturers and most concrete technicians, if not by the users of cement.

However, there appears to be some reluctance in admitting that these

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variations do exist, even under present day manufacturing technique. As the author points out, there is a tendency, whenever concrete failures occur or there is a lack of uniformity in concrete, to assume that the cement is above reproach.

The writer, during a recent discussion with a group of cement manufacturers, contractors and concrete producers, stated that tests made during the course of an investigation of one particular project indicated a variation in the strength and fineness of the cement of ±25 percent from the average strength and fineness found. Also, that these variations were generally reflected in the strength of the concrete. These findings were quite vigorously denied by some of the cement manufacturers present but the fact remains that these variations, determined by experienced testers in accordance with standard test methods, were found.

Current specifications for portland cement, with the exception of a few, cover only the characteristics of the finished cement. While it is true that most of the cement manufactured today is in accordance with specification requirements, it must be agreed that most of the specifications have such broad limits that different lots of cement may have widely varying characteristics and still be in accordance with the specification requirements.

The four million cu yd of uniformly, excellent quality concrete reported by Stadtfeld is an impressive total and should certainly lead other users to give consideration to more rigid control of the quality of the cement which they are purchasing. Further, the findings reported by Stadtfeld should be considered a refutation of certain claims which have been made that variations of cement quality within the specification limits have only a negligible effect on concrete.

It has been said by some authorities that bin testing of cement used in commercial ready-mix or transit-mix concrete imposes undue hardship on the cement manufacturers and concrete plants and is not practical. Regardless of the hardship imposed or the practicability of pretesting, this control should be insisted upon if uniform, high quality concrete is desired.

In addition, advance information regarding the quality of the cement to be used can result in an actual dollars and cents saving as it has been found that some cements require the use of as much as a full sack more per cubic yard of concrete to obtain the same strength and placeability.

Disagreement is found with the author's statement that no barge load of gravel can ever be sampled for grading after it arrives in the harbor. Under proper conditions of batch plant control it has been found that each barge load of aggregate can be sampled and tested prior to use so that necessary adjustments, due to grading variations, may be made to the mixture.

It is thought that many concrete technicians would be interested in knowing the procedure followed by the Board of Water Supply in batch plant control under present day conditions in the New York area. It is to be hoped that an article by Stadtfeld covering this phase of concrete inspection will be forthcoming at an early date.

AUTHOR'S CLOSURE

Subsequent to the publication of the paper and its presentation at the 1950 ACI Convention in Chicago, more than 70 requests have been made for Board of Water Supply, City of New York, specifications for portland cement. In addition many comments have been received from consumers. which leave the impression that there are still a number of cases where a consumer is not satisfied with the quality of the concrete produced. Unfortunately, all comments have been favorable and this, of course, is not a road toward progress, as some remarks by producers disagreeing with the main issue in the article would have been helpful towards an exchange of ideas.

The three discussions are also favorable. Blanks realizes that there are large variables in the production of portland cement and wishes the cement producers to be aware of it. Comack gives some details which may help in making efficient inspection.

Klein wishes to know our procedure in the matter of batch plant control in the New York area. With minor exceptions, we have always had our contractor install his own batching plant or let out the work to some other contractor who in turn would use the batching plant for our work only. The writer is therefore familiar with this procedure only and realizes that where more than one customer uses the same batching plant an additional difficulty is introduced to follow through one's own inspected materials.

Klein and the author disagree as to the procedure to be followed in sampling gravel for grading when gravel is delivered in barge load lots. There will always be differences of opinion, but to the writer's knowledge the only proper way for handling such a situation is to have an inspector present during the loading of the barge and have him require that the loading belt be stopped from time to time so that he can obtain a representative sample from the belt. In addition, he should be there to see that the properly graded gravel is loaded on the barge in such a manner as to prevent segregation as much as possible.

To conclude, since publication of the paper, the leach test mentioned in the paper indicating that certain Type I cements do not perform well in soft waters has been confirmed by another agency and some interest has been shown in inspecting the concrete which was claimed in the paper to show uniform good quality.

The author wishes to admit that there is one stretch of a high quality concrete road, about ⅛ mile long on a steep hill, where the upgoing lane is in bad condition although the downgoing lane is still perfect after ten years. This road was built with good aggregates, six bags of Board of Water Supply cement and one bag of natural cement containing tallow, an air-entraining agent. Distress was not noticed until after the fifth winter which was a severe one causing large quantities of calcium chloride to be used. Since the grade is rather steep no doubt the chains of heavy trucks using this road have ground in the calcium chloride and thereby helped to deteriorate the concrete in the upgoing lane. For the prevention of this condition the author knows no remedy.

Moments, shears and direct forces are determined for cellular shells by applying the least work theorem.

Flexure of Cellular Shells*

By F. E. WOLOSEWICK†

SYNOPSIS

After a brief discussion of the uses of cellular shells and methods available for the solution of design problems, a sample problem is set up using the theorem of least work. From charts the moments are determined and the effect of added stiffeners is assessed.

INTRODUCTION

Elongated cellular shells as shown in Fig. 1, with or without stiffening ribs, and subject to internal or external pressures, uniformly distributed, find considerable application in structural engineering. These shapes are used principally as caissons in building construction, and as hollow bridge piers. Under unusual circumstances they may function as silos and water towers.

Cellular shapes resist internal or external pressures by flexure of the shell, by direct stresses across the normal areas, and by shear across the normal sections. In limiting cases, when the shell becomes too thin it may collapse under the action of external forces.

Several methods are available for solutions of such problems: column analogy, method of work, and theorem of least work. In this paper, the theorem of least work was used to determine the necessary equations for moments and direct forces. Although column analogy is useful in investigating simpler cases, it does not lend itself readily to determination of general explicit equations.

Equations for moments, shears and direct forces at the center lines of horizontal and vertical axes are extremely lengthy for the more complicated caissons, involving in some instances as many as thirty parameters. Algebraic solutions of such equations are difficult since the various functions differ by small quantities, and do not lend themselves readily to slide rule calculations. Graphs are presented, plotted for

*Received by the Institute Mar. 22, 1949. Title No. 46-16 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 4, Dec. 1949, *Proceedings* V. 46, Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Apr. 1, 1950. Address 7400 Second Boulevard, Detroit 2, Mich.

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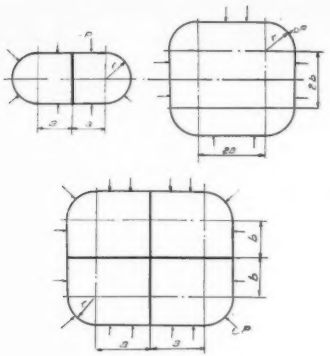

Fig. 1—Cellular shells with and without stiffening ribs

reasonable values of a/r and b/r for determining moments, shears and direct forces.

A critical section is always present when deformations are resisted by diaphragms and beams. Longitudinal shears and moments of high magnitude, but fortunately of localized character, are developed and are of the oscillatory type with decreasing amplitudes.

Such discontinuity stresses for circular sections have been extensively studied and solutions are available. For cellular shapes, with or without stiffening ribs, exact solutions are not known at present. Approximate analyses may be made by modifying circular shape constants by suitable factors. Selections of proper coefficients is a matter of judgment and past experience. The conventional one half of one percent of longitudinal reinforcing steel is sound policy.

The graphs in Fig. 2, 3, 4 and 5, give the necessary coefficients for determining forces and moments at certain critical points, for uniformly distributed pressures. For any given ratios of a/r and b/r forces and moments on center lines of sections can be determined. Once these forces are known, moments, direct forces and shears at other sections can be determined from statics by considering all forces to left or right of a section, whichever is more convenient.

The directions of all forces in the graphs are assumed as positive when acting as shown. For certain ratios of a/r and b/r, moments at centerlines become negative. It is then necessary to reverse directions of moments. All direct forces are for externally applied forces. For internal pressures, directions must be changed.

EXAMPLES

As an illustration consider a caisson whose length between radii is 27.75 ft along the a dimension, and the value of $2b = 10.25$ ft. The

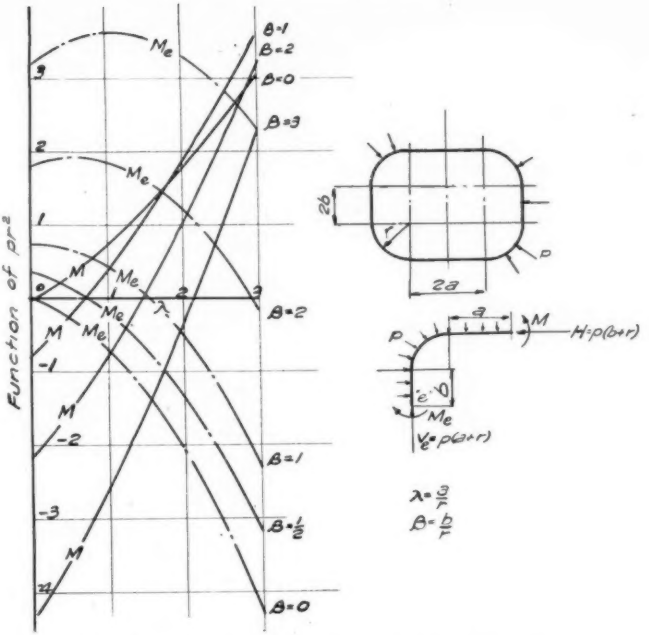

Fig. 2—Moments and shears in unstiffened cellular shells

radius is 5.0 ft, and the walls of the caisson are 18 in. thick for preliminary designs. The caisson is to be sunk to a depth of 80 ft, and the external pressure is assumed to be 70 lb per sq ft for each foot of depth. The problem is to determine moments throughout the shell.

At a depth of 80 ft the external pressure is approximately 5600 lb per sq ft. For a ratio of $a/r = 13.875/5.0 = 2.75$, and $b/r = 5.125/5 = 1.02$, moments in the unstiffened caisson can be obtained directly from Fig. 2. Thus $M_e = -1.80\ pr^2 = -1.80 \times 5600 \times 25 = -252{,}000$ ft lb while $M = 3.25 \times 5600 \times 25 = 470{,}000$ ft lb. Vertical force V_e is obtained directly from statics and is equal to $5600 \times (13.75 + 5) = 105{,}500$ lb, while H is equal to $5600 \times (5.125 + 5) = 56{,}000$ lb.

As shown in Fig. 2, the direction of M_e is assumed positive, or clockwise. This is also true for the value of M. For the ratios used in the above illustration, M_e came out negative, hence its direction must be reversed, or it must be acting in counterclockwise direction.

The values determined for the unstiffened caisson are too large to be handled by an 18 in. thick wall. By adding a wall connecting the hor-

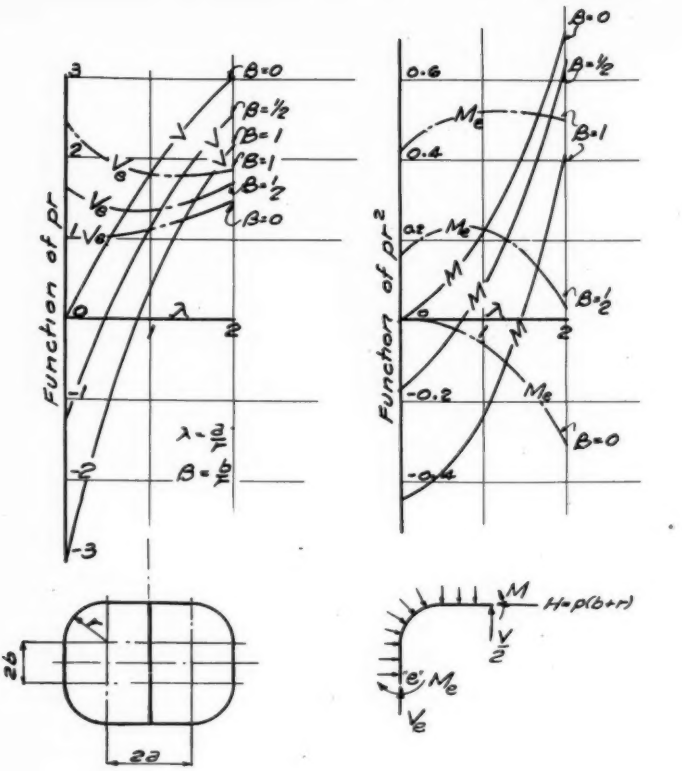

Fig. 3—Moments and shears in stiffened cellular shells

izontal walls together, moments may be decreased considerably. From Fig. 3, moment $M_e = 0.4 \times 5600 \times 25 = 56{,}000$ ft lb, positive, acting clockwise on point e. Moment M on the center line of $2a$ is out of range of the chart, but has been approximated at $1.60pr^2$. The value of $M = 1.60 \times 5600 \times 25 = 135{,}000$ ft lb. V_e is $2.20 \times 5600 \times 5 = 70{,}000$ lb, while the total reaction V in the wall is $2.60 \times 5600 \times 5 = 73{,}000$ lb, and H is the same as in the previous illustration.

For a doubly stiffened caisson shell with the same ratios as in the previous illustrations, direct forces and moments determined from Fig. 5 are as follows:

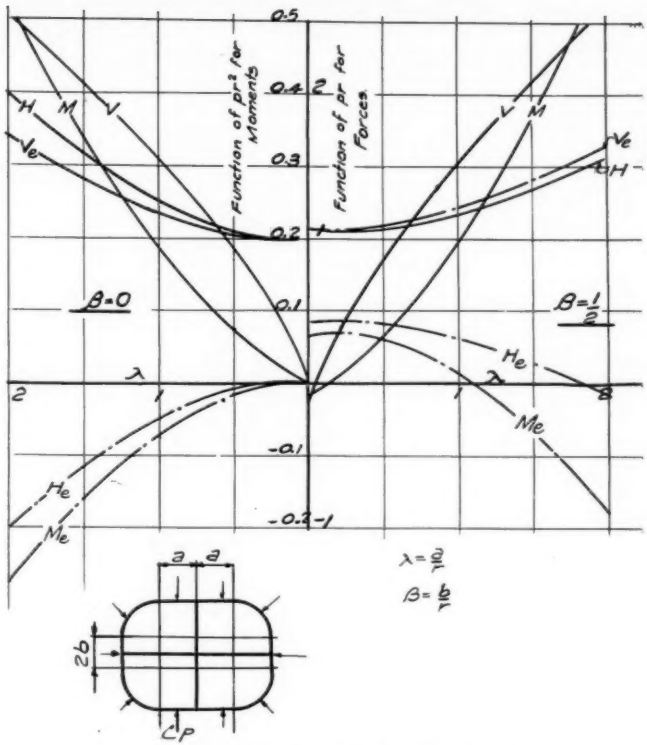

Fig. 4—Moments and shears in doubly stiffened cellular shells

$$
\begin{aligned}
M_e &= 0.3\times5600\times25 = 42{,}500 \text{ ft lb}\\
M &= 0.9\times5600\times25 = 125{,}000 \text{ ft lb}\\
H_e &= 0.8\times5600\times5 = 22{,}500 \text{ lb}\\
V_e &= 1.9\times5600\times5 = 53{,}500 \text{ lb}\\
V &= 3.0\times5600\times5 = 84{,}000 \text{ lb}\\
H &= 1.6\times5600\times5 = 45{,}000 \text{ lb}
\end{aligned}
$$

Since external pressures are assumed to vary linearly, moments and forces at other levels may be determined by proportion.

Equations for sections with different moments of inertia have also been evaluated, but because of their limited use in this type of work, were omitted from this article. It was discovered that for a caisson without a stiffening rib in the middle, by increasing moment of inertia over the length $2a$, moment M began to decrease, while moment M_e

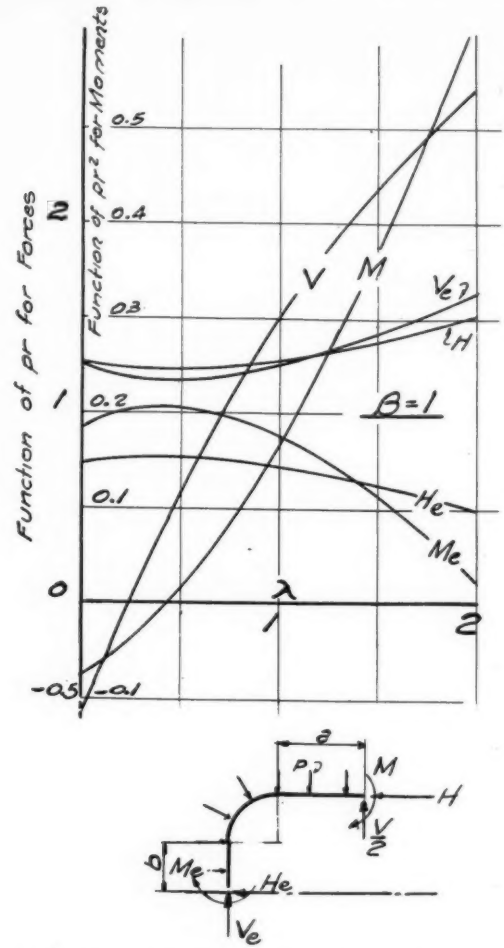

Fig. 5—Moments and shears in doubly stiffened cellular shells

began to increase. This conflicts with the accepted notion of rigid frames, that by increasing the inertia of the section, bending moments at that section are also increased.

For caissons with extremely sharp curvatures, pressures over the external surface are greater than those over the mean radius considered in developing the above equations. A study of such increased moments indicated that for a value of $b/r = 0$, and for a thickness of the wall equal to the radius, moments increased about $4\frac{1}{2}$ percent. For all practical purposes, such refinements may be ignored.

The Long-Time Study of Cement Performance in Concrete continues with a report on the performance of portland cements under exposure to sulfate soils.

Cement Performance in Concrete Exposed to Sulfate Soils*

By L. A. DAHL†

SYNOPSIS

The Long-Time Study of Cement Performance in Concrete deals with the performance of portland cements in concrete under various conditions of exposure in the field. Among these conditions is exposure to sulfate soils, that is, to the so-called "alkali soils." This part of the investigation is reported in Chapter 5*, in which the complete data are given. The present paper describes briefly the work reported in Chapter 5 and the conclusions which have been drawn. Those readers who wish to study the results in greater detail are referred to the more complete report.

CEMENTS

Twenty-seven test cements were used in the investigation of cement performance in concrete exposed to sulfate soils. The manufacture and properties of the cements are described in Chapters 1-4 of the Long-Time Study of Cement Performance in Concrete.‡ The calculated compound compositions and other data are given in Table 1. In each group the cements are arranged in order of descending values of C_3A. The specific surfaces of the cements and the C_3A values measured microscopically are included in the table.

EXPOSURE TO SULFATE SOILS

Two types of soil were used, each contained in a shallow concrete basin (36x48 ft and 10 in. deep) constructed in a plot near the laboratory of the Materials and Research Department of the California Division

*Received by the Institute Nov. 14, 1949. Title No. 46-17 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 4, Dec. 1949, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Apr. 1, 1950. Address 7400 Second Boulevard, Detroit 2, Mich.

This paper is a condensed version of Chapter 5, "Concrete Exposed to Sulfate Soils," of the Long-Time Study of Cement Performance in Concrete. Chapter 5, written by F. R. McMillan, T. E. Stanton, I. L. Tyler and W. C. Hansen, constitutes a special publication of the American Concrete Institute, published in cooperation with the Portland Cement Association, Dec. 1949 and available at ACI headquarters at $1.00 per copy; 60 cents to ACI Members.

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‡ACI JOURNAL, Feb., Mar., Apr., May 1948; *Proc.* V. 44, pp. 441, 553, 745, 877.

TABLE 1—CALCULATED COMPOUND COMPOSITION AND OTHER DATA ON CEMENTS

Cement No.	Calculated compounds*, percent by weight					Meas. value	Minor oxides, percent by weight				Specific surface A.S.T.M.
	C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	C_3A	Free CaO	MgO	Na_2O	K_2O	
Type I cements											
18	45.0	27.5	13.0	6.8	3.1	3.8	0.3	2.6	0.12	0.13	1750
12	45.0	28.0	12.6	7.3	2.6	5.6	0.1	3.1	0.28	0.40	1815
11	51.0	21.0	12.1	7.2	2.7	3.1	0.4	3.7	0.21	0.51	1820
15	65.5	9.0	12.0	7.5	3.3	8.6	0.4	0.8	0.08	0.23	1845
17	51.5	23.0	10.4	9.3	2.9	4.2	0.4	1.1	0.08	0.43	1770
13	50.5	26.0	10.0	6.5	2.8	1.6	1.6	1.1	0.04	0.19	1665
14	43.0	32.0	8.2	9.2	2.9	3.3	0.2	2.5	0.06	1.30	1880
16	53.0	21.5	7.4	10.7	2.9	4.4	0.7	2.1	0.23	0.46	1785
Type III cements											
31	56.5	16.0	10.7	6.4	3.8	3.2	1.5	3.3	0.23	0.22	2975
33	59.0	13.5	10.4	7.7	3.9	7.2	1.8	1.4	0.21	0.44	2530
34	64.0	10.5	5.6	10.0	2.9	4.2	2.3	2.5	0.28	0.28	2465
Type II cements											
22	41.5	33.0	6.6	11.7	2.4	2.2	0.1	3.2	0.24	0.37	1775
21	40.5	38.0	6.3	9.8	2.1	2.0	0.7	1.3	0.22	0.40	1630
24	41.5	28.5	5.4	14.8	3.0	2.9	0.9	3.1	0.05	1.30	1925
25	34.5	39.0	4.8	14.8	3.2	1.5	0.2	2.2	0.21	0.54	1825
23	51.5	22.5	3.8	16.6	2.6	1.2	0.4	0.9	0.59	0.14	1875
Type IV and V cements											
43	24.5	48.0	6.2	13.8	3.6	2.8	0.1	1.6	1.00	0.08	1965
43A	29.5	50.5	5.3	9.3	3.2	1.0	0.4	1.1	0.33	0.01	1915
41	20.0	51.0	4.7	15.2	3.4	1.5	0.4	3.0	0.06	1.19	1915
51	41.5	39.0	3.8	10.0	2.4	1.1	0.5	1.7	0.08	0.22	2025
42	27.0	55.0	3.4	8.2	2.6	0.9	0.2	1.8	0.16	0.26	1920
Air-entraining cements											
18T	45.0	28.5	13.2	6.6	2.9	3.8	0.4	2.6	0.14	0.13	1960
12T	45.0	28.0	12.5	7.1	2.6	5.6	0.1	3.1	0.31	0.40	1790
11T	51.0	21.5	12.2	7.1	2.6	3.1	0.5	3.7	0.22	0.51	2010
33T	56.5	16.5	10.4	7.4	3.7	7.2	1.8	1.5	0.21	0.46	2510
16T	51.5	22.5	7.9	10.4	3.1	4.4	0.8	2.0	0.23	0.44	1680
21T	38.5	40.0	6.6	9.8	2.2	2.0	0.6	1.3	0.21	0.43	1750

*Corrected for free CaO, not corrected for minor oxides. These figures are correct and are based on calculations from oxide analyses expressed to two decimal places. Figures in Table 3-4 of Chapter 3 were substantially the same as these except for some minor errors which escaped detection.

of Highways in Sacramento, Calif. This made it convenient for T. E. Stanton, a member of the Advisory Committee for the Long-Time Study of Cement Performance in Concrete, to closely observe the experiment. One soil was a natural soil very high in Na_2SO_4; the other was the same soil to which $MgSO_4$ and alkali-free soil were added to replace about one-third of the Na_2SO_4 with $MgSO_4$. Because of losses through various causes it was necessary to add Na_2SO_4 and $MgSO_4$ from time to time.

The natural soil in Basin 1 contained about 10 percent soluble material, mostly Na_2SO_4. After the first addition of $MgSO_4$ to Basin 2 the analysis of the salts in the two basins was as follows:

Soluble salts (Hypothetical combinations), percent by weight

	Basin 1	Basin 2
Sodium carbonate (Na_2CO_3)	9.77	1.66
Sodium chloride ($NaCl$)	1.46	3.14
Sodium sulfate (Na_2SO_4)	84.61	61.80
Magnesium sulfate ($MgSO_4$)	2.31	28.63
Calcium sulfate ($CaSO_4$)	1.52	3.43
Undetermined	0.33	1.34

Water from the basins was analyzed at irregular intervals, and salts were added as required. The percentages of Na_2SO_4 and $MgSO_4$ varied during the periods between additions.

Test beams, 6 x 5¼ x 34 in. made from three different mixes were laid horizontally in the soil, with half of the 6-in. dimension submerged and the troweled surface upward. Water was added at frequent intervals to just cover the upper surfaces of the specimens. In the summer rapid evaporation left heavy deposits on the soil and exposed portions of the beams. Winter rains kept water above the beams, sometimes overflowing from the basins.

The 27 test cements were used in each of the three concrete mixes, lean, medium and rich (4, 5½ and 7 sacks of cement per cu yd). For the non-air-entraining cements the corresponding percentages of sand, by absolute volume of total aggregate, were 45, 43½ and 42. A slump of 1½ in. was sought for each of the three mixes. With the air-entraining cements it was necessary to adjust the sand content to allow for the amount of air entrained and the reductions in mixing water. In each basin there were placed six beams made from each mix. The beams were arranged in the basins in six rounds, each round containing 81 beams representing the three mixes of each of the 27 test cements. The degree of sulfate attack was determined from time to time by visual rating of the specimens, by tests in flexure and compression, and by determination of natural frequency of vibration.

In addition to the beams exposed to sulfate soils, two control beams of each of the three mixes of the 27 cements were made for exposure to normal weathering. Concrete cylinders, 6x12 in., made from the same mixes, were cured for 28 days, then exposed to natural weathering. Some of these were tested for compressive strength at 28 days. The remaining cylinders were tested for absorption and compressive strength at the age of five years.

The beams were placed in the basins and the sulfate soils added to a depth of 3 in. during the last two weeks of December, 1942. Water was added to just cover the top surfaces of the beams. No evidence of

deterioration was observed during the first winter, but in late May and early June of 1943 when the salts were more concentrated because of evaporation, disintegration began to appear. By the end of October, 1943, after 10 months exposure, many specimens of the lean mix were completely disintegrated while others showed varying effects of sulfate action. The effect of C_3A content of the cement on the resistance to sulfate action was generally very marked. The appearance of the beams

Fig. 1a—Taken October 23, 1944, shows specimens in Basin 1 (soil predominantly sodium sulfate) after cleaning for inspection. Symmetrical distribution of the specimens in the basin by rounds and in numerical sequence of cements can be seen in this picture; specimens in almost perfect condition represent rich mix, those completely disintegrated the lean mix. Stakes shown in the picture separate successive rounds.

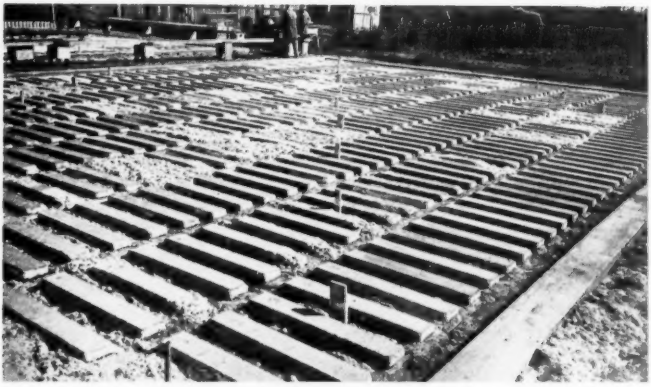

Fig. 1b—Similar view to Fig. 1a taken on same date across Basin 2 (soil mixed sulfates). There is generally less disintegration than in Basin 1

at this time may be seen in Fig. 1a and 1b. It may be seen that disintegration was greater in Basin 1 than in Basin 2.

MEASUREMENT OF RELATIVE RESISTANCE TO SULFATE ATTACK

The relative resistance of the beams to sulfate attack was measured by two methods: (1) rating based upon visual examination; and (2) strength and dynamic frequency tests at various intervals.

Rating by visual examination of the beams has been found satisfactory. This rating method could become of little value if different observers tend to differ in their ratings, and if a single observer tends to change his standards from time to time. To avoid these difficulties, a series of photographs representing successive degrees of attack were prepared for reference, ranging from a rating of 1 for unattacked specimens to 6 for those completely disintegrated, in steps permitting intermediate ratings. Figure 1c gives an idea of the degree of attack in specimens rated in whole numbers. At least two observers rated the specimens independently, in September or October of each year. Ratings in fair agreement were averaged for the record. Ratings were made jointly and agreed upon when there were significant differences in the separate ratings.

The second method of measuring resistance to sulfate attack has the advantage of substituting direct measurement of strength and dynamic frequency for visual inspection. It also gives some idea of the significance and consistency of the ratings. It was not considered practicable to remove all specimens from the basins each year for these tests. Instead, one round was removed from each basin at each annual inspection period.

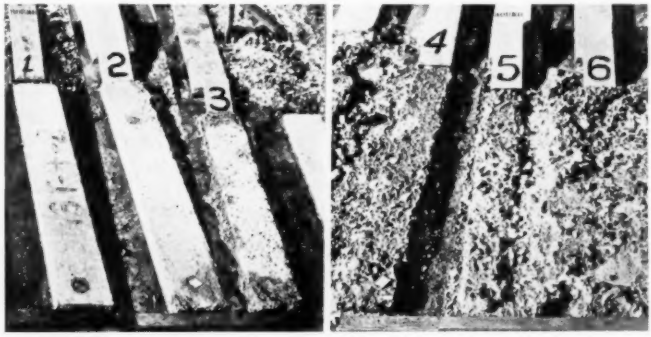

Fig. 1c—Shows the key used in rating specimens. Original photographs of these specimens with others taken at the same time showing fractional ratings were used in rating all specimens.

Continuation of this procedure would terminate the tests upon removal of the sixth round, and would leave no exposed specimens for ratings or for other tests at later periods. To avoid this situation, removal of specimens was discontinued when the fourth round was removed and tested. Specimens in the remaining two rounds are given ratings periodically and will be subjected to such tests as may be made with the specimens in place, until all are completely disintegrated.

Attention may be called here to laboratory determinations of the expansion of 1x1x11¼-in. prisms of 1-6 standard Ottawa sand mortars made from the 27 test cements during immersion in three different sulfate solutions. These tests were reported by Lerch and Ford in Chapter 3* of the Long-Time Study of Cement Performance in Concrete. The solutions were as follows: 0.15M Na_2SO_4; 0.15M $MgSO_4$; and 0.25M Na_2SO_4 + 0.25M $MgSO_4$. In these tests the expansions in the first two solutions did not differ significantly for individual cements. The more concentrated solution of the two salts was much more aggressive. All of the specimens made with the non-air-entraining cements of Types I and III disintegrated in this solution in less than two weeks. The general trend was toward decreasing expansion with decreasing calculated C_3A content, although there were some departures from the general trend. The relationship found was no better when C_3A percentages determined petrographically (see Chapter 4†) were used instead of the calculated values.

RATINGS OF SPECIMENS

Numerical ratings of the test beams at various periods are given in a series of tables and figures in Chapter 5. For the purpose of this condensation of that chapter it is sufficient to present the data for 10 months exposure and for the latest inspection date, at 69 months, in a single table. These data, in Table 2, give a comparison of conditions at an early period, before the specimens for one type of cement were completely disintegrated, with those at the latest period for which data are available. The ratings at 10 months are averages for six specimens. Because of removal of specimens for test, the ratings at 69 months are averages for two specimens.

In examining the data in Table 2 it should be remembered that a rating of 1 represents a perfect specimen and a rating of 6 represents complete disintegration. Each of the three-column sections in the table show the increasing resistance to sulfate attack with increasing cement content. At 10 months the 7-sack mix shows no evidence of attack except for three cements attacked so slightly that their rating is 1.1.

*ACI JOURNAL, Apr. 1948, *Proc.* V. 44, p. 745.
†ACI JOURNAL, May 1948, *Proc.* V. 44, p. 877.

TABLE 2—RATINGS OF SPECIMENS AT 10 AND 69 MONTHS

Cement No.	October 1943 (10 months)						September 1948 (69 months)					
	Basin 1			Basin 2			Basin 1			Basin 2		
	Sacks per cu yd			Sacks per cu yd			Sacks per cu yd			Sacks per cu yd		
	4	5.5	7	4	5.5	7	4	5.5	7	4	5.5	7
Type I cements												
18	6.0	3.4	1	4.0	1.5	1		6.0	5.8	6.0	6.0	4.4
12	6.0	3.7	1	2.3	1	1		6.0	1.9	6.0	6.0	2.0
11	6.0	2.9	1	3.1	1	1		6.0	1.3	6.0	6.0	2.0
15	6.0	2.7	1	2.7	1	1		6.0	1.9	6.0	6.0	2.0
17	6.0	2.2	1	1.9	1	1	a	6.0	2.5	5.7	3.1	2.4
13	5.7	3.0	1	2.1	1	1		6.0	1.5	5.9	3.3	1.7
14	5.1	2.4	1	2.1	1	1		5.8	1.3	4.6	2.2	1.8
16	4.9	2.4	1	2.0	1	1		6.0	2.6	3.3	2.5	1.8
Av.	5.7	2.8	1	2.5	1.1	1		6.0	2.3	5.4	4.4	2.3
Type III cements												
31	6.0	1.4	1	1.6	1	1		6.0	1.9	6.0	3.3	2.1
33	6.0	1.5	1	2.0	1	1	a	6.0	1.3	6.0	2.2	1.9
34	4.6	1.9	1	1.5	1	1		5.2	1.2	2.9	1.9	1.8
Av.	5.5	1.6	1	1.7	1	1		5.7	1.5	5.0	2.5	1.9
Type II cements												
22	5.0	3.1	1	2.5	1	1		6.0	1.7	5.5	2.4	2.0
21	4.1	2.4	1	2.1	1	1		4.3	1.5	3.8	1.8	1.8
24	4.5	2.2	1	2.0	1	1	a	4.2	1.2	3.3	1.8	1.8
25	4.5	2.5	1	2.1	1	1		4.5	1.3	3.2	1.9	1.7
23	4.2	2.4	1	1.7	1	1		4.4	1.2	3.0	1.7	1.9
Av.	4.5	2.5	1	2.1	1	1		4.7	1.4	3.8	1.9	1.8
Types IV and V cements												
43	4.0	2.2	1.1	2.0	1	1	5.6	3.9	1.3	3.1	1.8	1.8
43A	4.0	2.0	1.1	2.6	1	1	5.5	2.4	1.2	3.5	2.0	1.8
41	4.2	2.4	1	2.1	1	1	5.3	4.2	1.6	3.5	1.9	1.7
51	4.1	1.9	1.1	2.6	1	1	5.5	2.6	1.2	3.3	1.9	1.7
42	4.0	2.5	1.1	2.5	1	1	4.9	3.2	1.6	3.3	1.9	1.8
Av.	4.1	2.2	1.1	2.4	1	1	5.4	3.3	1.4	3.3	1.9	1.8
Air-entraining cements												
18T	5.5	2.1	1	2.7	1	1	6.0	6.0	4.3	6.0	6.0	3.3
12T	5.8	2.4	1	2.3	1	1	6.0	6.0	2.3	6.0	6.0	2.8
11T	4.8	2.2	1	1.7	1	1	6.0	6.0	1.4	5.8	3.5	1.8
33T	4.2	1.3	1	1.5	1	1	6.0	6.0	1.2	4.6	1.9	1.6
16T	2.7	1.0	1	1.9	1	1	5.5	1.6	1.2	2.7	1.7	1.7
21T	2.8	1.2	1	2.3	1	1	5.1	1.6	1.2	3.7	1.6	1.7
Av.	4.3	1.7	1	2.1	1	1	5.8	4.5	1.9	4.8	3.4	2.1

[a]All specimens for Types I, II and III were completely disintegrated at 17, 58 and 20 months, respectively.

For the 4-sack mix there are seven cements with a rating of 6.0, indicating complete disintegration. At 69 months the greater resistance of the 7-sack mix to sulfate attack is still apparent. It is to be expected that the data on this mix will be more informative after a longer period.

Comparing the ratings for Basins 1 and 2 at 10 months it is seen that for the 4- and 5.5-sack mixes the ratings for specimens in Basin 2 are lower in every case than those for Basin 1, indicating that at that period the attack on specimens in the soil of Basin 1 (predominantly Na_2SO_4) was greater than in Basin 2, in which $MgSO_4$ replaced about one-third of

the Na_2SO_4. At 69 months the differences in ratings of corresponding specimens in the two basins is generally in the same direction, although not as consistent nor as great. For the 4- and 5.5-sack mixes the average ratings in Basin 2 are 1.3 and 1.8 points lower, respectively, than in Basin 1. For the 7-sack mixes the average difference is negligible, the average rating for Basin 2 being only 0.2 points higher than for Basin 1. The greater effect of Na_2SO_4 on the lean and intermediate mixes would not be expected from the laboratory tests on the mortar prisms. An explanation is offered in an appendix to Chapter 5.

The influence of C_3A content of the cement on resistance to attack by sulfate soils is shown in Fig. 2. In this figure the dates selected are those which give a range of ratings from low to high throughout the entire range of C_3A values, without having too many perfect specimens at early periods, or too many completely disintegrated specimens at later periods. For three of the diagrams the date was that of the latest inspection (September, 1948). In two of these, for the 7-sack mixtures, it is apparent that the range of ratings is too small to give an indication of the influence of C_3A content. In these cases it is possible that after further exposure for a few years the influence of C_3A content will be more readily apparent.

From Fig. 2 it may be seen that the C_3A content has a large influence upon resistance of the 4- and 5.5-sack mixes to sulfate attack. Variations from the trend indicate that other factors are also involved.

TESTS OF SPECIMENS AFTER EXPOSURES OF 1 TO 5 YEARS

At intervals of approximately one year, one round of beams was removed from each basin, resonant frequencies were determined, and the beams were then tested for flexural and compressive strength (modified cubes on beam ends). The beams in Basin 2 were not removed for test at the end of the first year because disintegration was less pronounced than in Basin 1. The original plan of removing all specimens each year for determination of resonant frequency was not followed because it would require rehandling the entire amount of soil at each test period. As mentioned previously, removal of specimens was discontinued after four rounds had been removed, to give opportunity for continuing ratings of specimens and for such tests as can be made with the specimens in place.

Tables giving data on test values and ratings for individual specimens are given in Chapter 5 of the general report, but are too voluminous to be repeated here. Averages for each type of cement are given in Tables 3 and 4, and are shown graphically in Fig. 3 and 4. In the tables and figures the 4-, 5.5- and 7-sack mixes are designated as Mixes 1, 2 and 3, respectively. In each of the small diagrams in the figures the

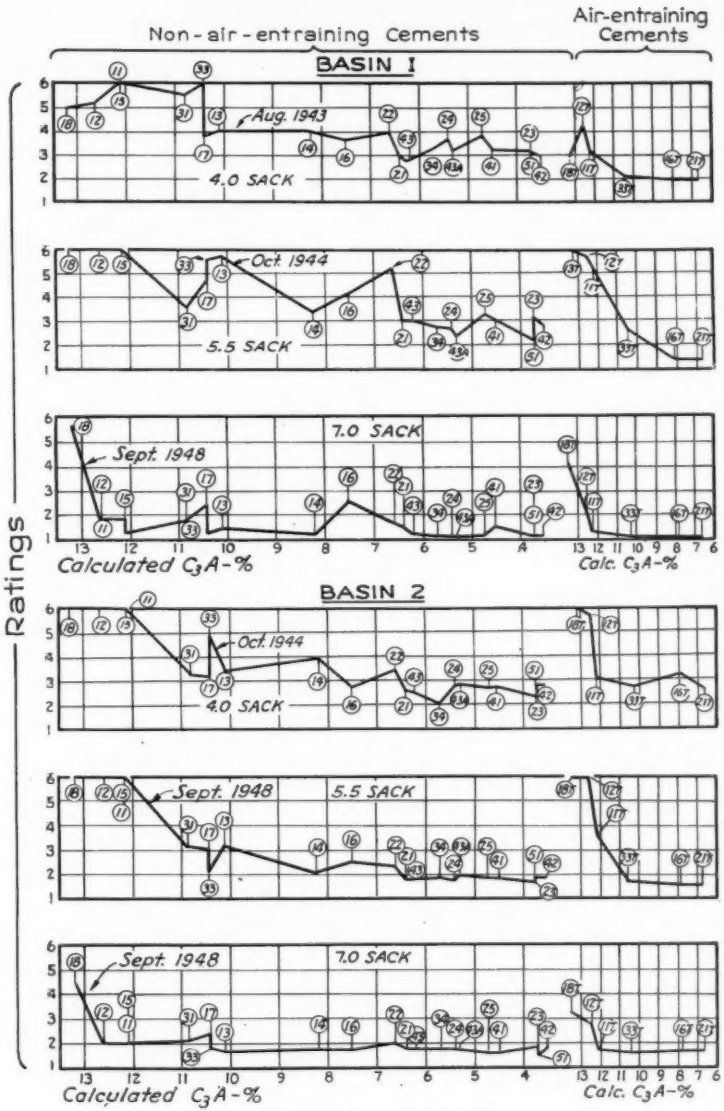

Fig. 2—Ratings versus calculated C_3A

TABLE 3—TESTS OF SPECIMENS AFTER 1 TO 4 YEARS EXPOSURE*
Average Results—Basin 1—Sodium Sulfate Soils

Age at test, years	Rating of specimens at time of test			Modulus of rupture, psi			Compr. strength, psi			Resonant freq. N^2, percent of orig.		
	Mix. No.			Mix No.			Mix No.			Mix No.		
	1	2	3	1	2	3	1	2	3	1	2	3
Type I cements												
1	5.7	2.6	1.1	0	826	970	0	5140	6980	0	108	107
2	6.0	5.5	1.3	0	30	1006	0	1160	7000	0	10	102
3	6.0	5.9	1.4	0	0	852	0	0	8340	0	0	101
4	6.0	5.9	1.4	0	0	701	0	0	6200	0	0	85
Type III cements												
1	5.5	1.8	1.0	0	660	1007	0	5285	6790	0	97	103
2	6.0	3.9	1.3	0	313	911	0	3380	6240	0	53	99
3	6.0	5.7	1.1	0	0	1005	0	870	7430	0	0	97
4	6.0	5.0	1.3	0	43	675	0	1490	5470	0	13	96
Type II cements												
1	4.5	2.7	1.0	163	863	1136	3056	5590	7480	52	105	109
2	5.4	3.8	1.3	8	533	1093	1203	4593	7646	7	78	105
3	6.0	3.7	1.1	0	172	1090	0	4120	9620	0	55	103
4	5.8	3.9	1.1	0	132	905	0	3965	7930	0	36	100
Types IV and V cements												
1	4.0	2.5	1.2	750	970	990	4940	6290	7140	127	145	115
2	4.4	2.8	1.3	285	960	1140	4400	6935	7725	73	112	111
3	4.5	2.6	1.1	103	820	1070	3907	6507	9635	68	112	111
4	4.3	3.0	1.2	219	676	827	2660	7820	7626	31	99	106
Air-entraining cements												
1	4.0	1.4	1.0	110	850	980	2120	5360	6780	19	106	110
2	5.1	3.9	1.3	50	380	920	990	2770	6580	14	53	108
3	6.0	3.8	1.1	0	325	866	0	2860	8955	0	35	107
4	5.8	4.2	1.4	0	243	703	0	1843	6366	0	36	102

*No specimens removed from basin for test after 4 years.

graphs for the three mixes are designated by the mix numbers in circles. In general there is a marked decrease in strength and elastic modulus (N^2, percent of original)* for the lean and intermediate mixes during the period involved. This decrease is more pronounced for cements of Type I and III than for those of Type II, IV and V, and is more pronounced in Basin 1 than in Basin 2. For the rich mixes (Mix 3, 7 sacks per cu yd) the strengths of specimens in Basin 1 show increases, or only a slight decrease between the first and third years, but generally show a decrease between the first and fourth year. The data on specimens from Mix 3 are not as consistent as for the other two mixes, probably because a longer time is required to bring out differences in the performance of these specimens. It is of interest to note that changes in ratings during the period are generally consistent with the other indications.

*For specimens of a given size and shape the modulus of elasticity is proportional to the square of the resonant frequency, N. In the tables and figures the square of the resonant frequency of each specimen is therefore represented as a percentage of the square of the resonant frequency of the specimen before exposure to sulfate attack.

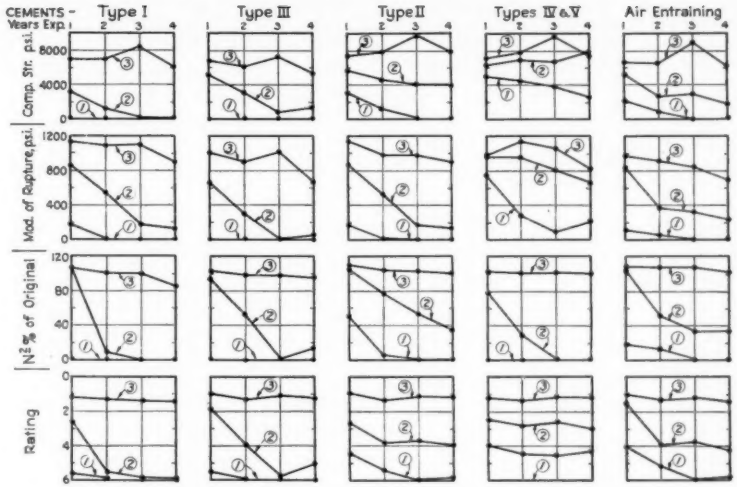

Fig. 3—Average results of tests after 1-4 years exposure—Basin 1

For the specimens in Basin 1 the changes in resonant frequency, shown in Table 3 and Fig. 3, are quite consistent with the changes in ratings and in flexural and compressive strength. The graphs for resonant frequency in Fig. 4, for Basin 2, show similar trends, consistent with strengths and ratings, but do not show as large changes in resonant frequency as those in Fig. 3.

The general agreement of the visual ratings with the data on strengths and resonant frequency is of considerable interest, since ratings after removal of the fourth round will provide data on the condition of the specimens at later periods. This applies particularly to specimens made from the rich mix, as these specimens have been attacked much less severely than those from the lean and intermediate mixes.

EFFECT OF AIR ENTRAINMENT

In this brief review only a very small part of the data on individual specimens can be presented. However, the complete data given in Chapter 5 of the Long-Time Study of Cement Performance in Concrete show clearly that specimens made with air-entraining cements are more resistant to sulfate attack than those made with the corresponding non-air-entraining cements. The comparison shown in Fig. 5 is sufficient to indicate the general effect of air entrainment, as judged by visual ratings. In this figure it is apparent that specimens made with the air-entraining cements generally have lower ratings, and are consequently

TABLE 4—TESTS OF SPECIMENS AFTER 2 TO 5 YEARS EXPOSURE*
Average Results—Basin 2—Sodium and Magnesium Sulfate Soils

Age at test, years	Rating of specimens at time of test			Modulus of rupture, psi			Compr. strength, psi			Resonant freq. N^2, percent of orig.		
	Mix No.			Mix No.			Mix No.			Mix No.		
	1	2	3	1	2	3	1	2	3	1	2	3
Type I cements												
2	4.7	2.6	1.4	267	724	954	1555	4735	7762	26	75	101
3	4.9	3.1	1.4	116	555	995	1670	4335	7590	25	74	101
4	5.2	4.1	1.6	26	385	920	465	2920	7526	11	50	100
5	5.4	4.0	1.8	55	397	1103	453	2560	6734	18	60	102
Type III cements												
2	3.4	1.3	1.4	505	880	902	4170	6618	7750	34	96	99
3	4.3	1.3	1.4	167	826	970	2835	6160	—	32	97	101
4	4.2	1.9	1.6	220	702	872	1507	6065	7193	32	98	99
5	3.7	1.6	1.6	200	902	1003	2618	5580	6111	31	98	98
Type II cements												
2	2.5	1.2	1.5	482	913	1055	4640	6380	8015	83	101	102
3	2.7	1.4	1.2	324	878	1109	4487	6211	—	82	104	103
4	2.6	2.1	1.6	498	852	1050	3875	6004	7890	94	102	102
5	3.2	1.7	1.5	303	817	1235	3226	5741	5492	77	101	105
Type IV and V cements												
2	3.2	1.8	1.4	868	1040	1025	4772	6986	8030	133	107	106
3	2.3	1.4	1.3	760	884	1089	4950	6790	—	116	108	109
4	3.0	2.1	1.5	743	961	927	4750	7073	7654	119	107	106
5	2.9	1.7	1.6	703	1047	1252	4313	6116	7447	120	112	108
Air-entraining cements												
2	3.6	2.2	1.6	380	820	920	2780	5870	7470	53	100	100
3	3.9	2.6	1.3	220	550	960	2004	3800	—	49	67	101
4	4.6	3.2	1.8	56	470	830	1370	3880	7120	26	67	100
5	4.3	2.8	1.7	248	560	960	1670	4270	6220	47	79	104

*No specimens removed from basin for test at end of first year.

in better condition, than the specimens made with corresponding non-air-entraining cements, when inspected in the months indicated. Those made with Mix 3 (7 sacks per cu yd) show little difference, as might be expected because of their greater resistance to sulfate attack.

It should be noted that the air contents of the air-entraining cements were generally lower than those usually sought for resistance to the action of sodium and calcium chlorides on pavements. It is possible that higher air contents would have led to greater differences in ratings than those shown in Fig. 5.

C_3A AS A CRITERION OF SULFATE RESISTANCE

In each of the diagrams in Fig. 2 there is a general trend toward lower ratings (greater resistance to sulfate attack) with decreasing C_3A content. Departures from the general trend are to be expected, since it is not to be supposed that the C_3A content is the only variable influencing resistance to sulfates. However, the trend is sufficiently marked to indicate that the C_3A content, as calculated, may be of considerable value

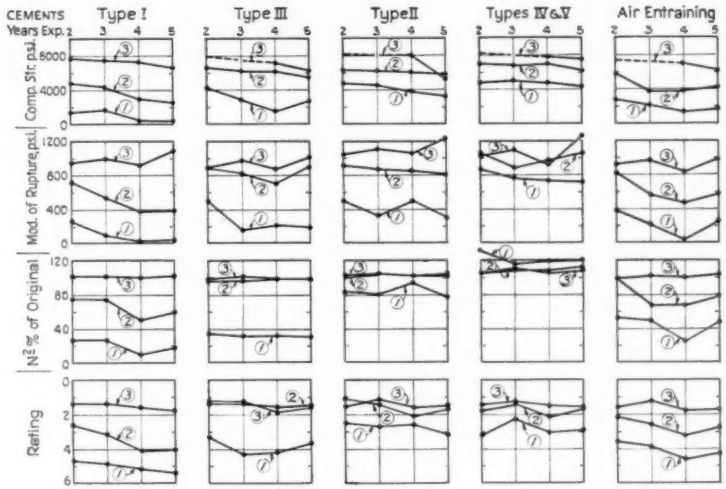

Fig. 4—Average results of tests after 2-5 years exposure—Basin 2 (Dotted lines indicate intermediate values are lacking)

in distinguishing between cements which may be expected to be resistant to attack by sulfate soils and those which may not.

The upper diagram of Fig. 2 is of no value in estimating a value of C_3A which may be taken as a dividing point between good and poor sulfate resistance, since the data in this figure, representing only about 8 months exposure in Basin 1, show that disintegration has occurred to a considerable extent in all specimens. On the other hand, the third and sixth diagrams, for the 7-sack mixtures at about six years show considerable resistance to attack at that time, with one exception. The second and fourth diagrams, indicating ratings after two years of exposure, will therefore be considered with reference to non-air-entraining cements. In both diagrams a C_3A content of 7 percent appears to be a satisfactory point of division. Cements with less than 7 percent C_3A generally have lower ratings than those with more than 7 percent C_3A. It is not a perfect criterion, however, since there are exceptions. Cement 22 is the most glaring exception, since in the second diagram it has a rating of 5.2, and yet would be accepted on the basis of its C_3A content. In the fourth diagram Cement 22 has nearly the same rating as Cements 13 and 17, although the latter cements would be rejected on the basis of their C_3A content.

Among the cements with less than 7 percent C_3A there is no marked differences in the ratings of specimens, in either Basin 1 or Basin 2. That

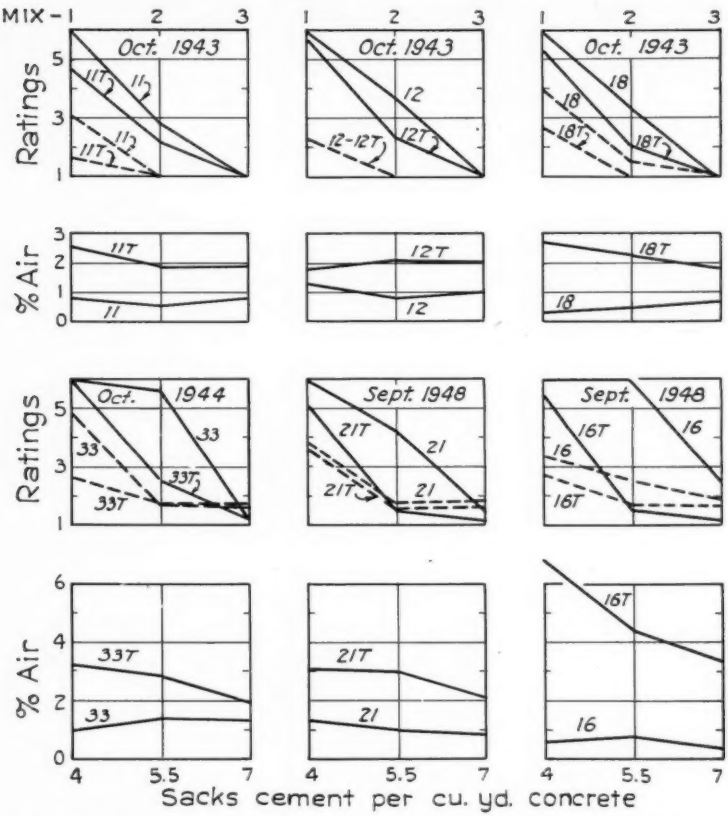

Fig. 5—Comparison of air-entraining and non-air-entraining cements (Solid line—Basin 1; Dash line—Basin 2)

is, a separation of this group with reference to another C_3A content, say 5 percent, will not distinguish those of superior performance (low ratings). The effect of choosing 5 percent C_3A as a dividing line, instead of 7 percent, would therefore be to reject cements of good sulfate resistance which would be accepted on the basis of the 7 percent limit.

With one exception, the 7-sack mixtures shown in the third and sixth diagrams give evidence of only slight attack by sulfate soils after about 6 years exposure. The differences in ratings are not great enough to give an indication of the influence of C_3A content. Upon comparing these diagrams with those for the 4- and 5.5-sack mixtures at earlier

periods it is evident that the use of rich mixtures is of prime importance in securing maximum sulfate resistance. As stated previously, a considerably longer period of exposure will be required to determine the influence of C_3A content on the sulfate resistance of the 7-sack mixtures.

The value of C_3A content as a criterion of sulfate resistance has been considered with reference to the second and fourth diagrams in Fig. 2, In these diagrams the ratings indicate considerable sulfate attack, and this is true even in the case of cements below the suggested limit of 7 percent C_3A. The mixes involved are evidently too lean for satisfactory sulfate resistance. The real test of C_3A content as a criterion will be in the 7-sack mixtures at some future time when the influence of C_3A content becomes evident. It is probable that the results at that time will be of the same order as those which have been found for lean and intermediate mixes. Although this is probable it is not a certainty. To illustrate, the diagrams for Basin 1 may be compared with the corresponding diagrams for Basin 2. Taking into account the times at which the ratings were made, it is seen that in the case of the 4- and 5.5-sack mixes sulfate attack was much more rapid in Basin 1 than in Basin 2, as previously mentioned. On the other hand, this situation is reversed in the 7-sack mixes, sulfate attack being greater in Basin 2 than in Basin 1. That is, the behavior of the rich mix can not be predicted from the behavior of the lean and intermediate mixes. It will be interesting to follow the progress of the 7-sack mixes in later years of exposure to the sulfate soils.

EXPANSION OF MORTAR BARS AS A CRITERION OF SULFATE RESISTANCE

Laboratory tests in which mortar bars are stored in solutions of Na_2SO_4, $MgSO_4$ or in a mixture of the two offer some promise of providing a means of determining whether a cement may be expected to be resistant to sulfate soils. Expansion of the mortar bars at a specified period might be taken as a criterion of resistance to sulfate soils if a relation between them is observed. A limit of 0.15 percent on expansion of mortar bars in 0.15M Na_2SO_4 solution at 3 weeks, or 0.25 percent at 4 weeks, makes exactly the same separation of the 21 non-air-entraining cements as the C_3A limit of 7 percent just discussed. That is, the relation between C_3A content and the expansion of mortar bars in the Na_2SO_4 solution is such that there is no choice between selecting the C_3A content and the expansion of mortar bars in Na_2SO_4 as a criterion of resistance to sulfate soils. Each criterion possesses the same virtues and the same faults. The remarks which have been made previously concerning the importance of judging the value of C_3A as a criterion by the behavior of 7-sack mixtures after further exposure apply also to the expansion of mortar bars as a criterion.

CONCLUSIONS

1. Resistance of concrete to attack by sulfate soils has been found to be principally related to (a) the cement content of the concrete and (b) the calculated tricalcium aluminate (C_3A) content of the cement.

For the rich mixes (7 sacks of cement per cu yd concrete) there is very little difference in the condition of concrete specimens after six years' exposure. The effect of cement composition is more readily observed in the 4- and 5.5-sack mixtures, which were attacked much more rapidly. It is likely that at six years the increased impermeability of the richer mix, owing to the lower water-cement ratio, has resulted in high resistance to sulfate attack even with cements of high C_3A content.

Several marked departures from the trend in the relation of C_3A content to the ratings of specimens exposed to sulfate soils indicate that other factors are involved. These will be given further study.

2. Air entrainment was found to lead to increased sulfate resistance, notwithstanding the fact that the air contents were generally below those recommended for concrete pavements subjected to applications of sodium and calcium chlorides. The effect of air entrainment might have been greater if higher air contents had been employed.

3. The C_3A content of the cement and the expansion of 1-6 mortar bars in Na_2SO_4 solution were of equal significance in evaluating the performance of the cements in concrete exposed to sulfate soils. This conclusion is based upon the data obtained with lean and intermediate mixtures. The specimens made with the 7-sack mix have not been exposed for a sufficient time to permit conclusions in regard to the value of these criteria with reference to the richer mix.

The advantages of steam curing appear to be limited to the early age strengths of concrete as shown by recent tests.

Early Strength of Concrete as Affected by Steam Curing Temperatures*

By JOSEPH J. SHIDELER† and WILBUR H. CHAMBERLIN†

SYNOPSIS

The testing and results obtained on 990 6 x 12-in. concrete cylinders steam cured for various periods and at temperatures ranging from 100 to 200 F are discussed. Strength results are given for ages ranging from 6 hours to 28 days and strengths of companion specimens moist cured at 70 F are given whenever possible. The information presented was obtained for the Bureau of Reclamation for writing specifications covering the steam curing of precast units such as irrigation pipe.

INTRODUCTION

The tests described were made to obtain information on the use of steam to accelerate strength development and to provide moist curing at temperatures between 100 F and 200 F as normally used in the manufacture of concrete pipe. These tests were designed to furnish data on the compressive strength of concrete as affected by steam curing temperatures, length of curing, age at which steam curing was started, and the possible gain in strength after a period of steam curing.

The results indicate that 28 days of continuous fog curing produced higher 28-day compressive strengths than any of the steam curing or fog and steam curing combinations and that the apparent advantages of steam curing appeared to be limited to the early age strengths. The results also showed that a delay of a few hours prior to steam curing was beneficial; that temperatures between 130 F and 165 F produced optimum results; that temperatures above 165 F and rapid temperature rise were injurious to the concretes; and that the advantage of continued steam curing was very marked for the first 24 hours but decreased rapidly with age and had little value after about 48 hours.

*Received by the Institute Sept. 12, 1949. Title No. 46-18 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 4, Dec. 1949, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Apr. 1, 1950. Address 7400 Second Boulevard, Detroit 2, Mich.

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Three series of tests were made to obtain compressive strength measurements on 6 x 12-in. concrete cylinders tested through a wide range of steam room temperatures and various periods of steaming.

The first series of tests consisted of steam curing the cylinders immediately after they had been cast; in the second series the initial steam curing was followed by fog curing at 70 F; and in the third series the specimens were subjected to preliminary fog curing at 70 F prior to steam curing. Fog-cured control cylinders were cast with each of the three series. The first two series included Types I, II and V cement; in the third series Type V was omitted due to lack of material.

TEST PROCEDURE

A study of data obtained from preliminary tests on 6 x 12-in. concrete cylinders cast in steel open-top molds and raised to maximum steam room temperatures of 190 F and 200 F in a minimum of time indicated that these high temperatures or the rapid temperature rise had a very detrimental effect on the structure and strength of the concrete. With this information as a basis of what might be expected, the following three series of tests were outlined and performed to obtain a more comprehensive view of concrete behavior as manifested by compressive strength measurements on a large number of cylinders subjected to standard and elevated curing temperatures.

The first series consisted of testing approximately 390 cylinders from 15 mixes containing Types I, II, and V cement. Within 1 hour after casting, each set of specimens was subjected to one of five different steam room temperatures ranging from 100 to 195 F for periods of 3 hours to 7 days and were tested shortly after their removal from the steam room. The advantages of a "soaking" period, in which the temperature of the specimen is lowered slowly after the steam is turned off, as discussed by Mansfield,[1] are recognized, but this cycle could not be readily applied to these tests. The second series of tests included approximately 130 cylinders from 10 mixes containing Types I, II and V cement. These specimens were also steam cured immediately after casting at temperatures of 100, 130, 165 and 185 F. After steam curing for periods of 6, 24 and 72 hours, the specimens were removed from the steam room, part of them tested, and the remaining specimens were transferred to the standard curing room for testing at 7 and 28 days. The third series of tests contained approximately 176 cylinders from 8 mixes made with Types I and II cements. Steam room temperatures were identical to those used in the second series of tests. The specimens were fog cured for periods of 0, ½, 1, 2, 4, 6, 8 and 12 hours prior to steam curing; 24 hours after casting, all specimens were removed from the steam-curing

room and tested. Each group of tests included standard-cured control specimens tested at several early ages and at 28 days.

The aggregate grading, mix proportions and materials used for the three series of tests were approximately the same and are shown in Table 1. Sand and gravel for the mixes were batched by weight and were mixed for 5 minutes in a 6½ cu ft rotating drum-type mixer.

Steam curing was accomplished in a specially built room equipped with three steam outlets and containing ample space to permit free circulation of air about the specimens. Copper sheeting on the walls and ceiling acted as a moisture barrier, and a thermostat controlled the temperature within ±2 F. An electric resistance thermometer was placed in a special cylinder cast with each set of cylinders, and the temperature within the specimen and the ambient temperatures were continuously recorded. In all tests, the specimens attained the ambient temperature in about 1 hour and 15 minutes. Steam room and concrete specimen temperatures are plotted in Fig. 1 for three of the tests. All heating was by the injection of saturated steam. The steam pressure at the boiler was 180 psi, but was reduced through a regulating valve to 10 psi at the steam room outlet.

DISCUSSION OF TEST RESULTS

General

Inspection of the data reveals a marked variation in the early strengths of concrete specimens obtained from identical mixes, but the coefficient of

TABLE 1—MIX DATA

Cement: Portland, Type I
Type II
Type V
Aggregate: Platte River
Mix (parts by weight): 1:2.42:3.94
W/C ratio: 0.55
Slump: 3 in.
Cement content (lb per cu yd): 515
Percent sand: 38
Water content (lb per cu yd): 282

Aggregate grading

Screen size	Individual percent retained	Cumulative percent retained
Sand		
No. 8	15	15
No. 16	15	30
No. 30	25	55
No. 50	24	79
No. 100	16	95
Pan	5	100
Fineness modulus 2.74	0.6 percent absorption factor	
Gravel		
No. 4- ⅜ in.	22	22
⅜- ¾ in.	33	55
¾-1½ in.	45	100
Fineness modulus 6.77		

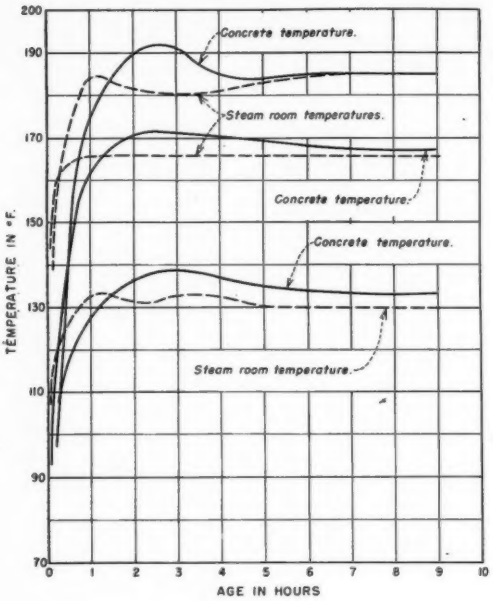

Fig. 1—Steam room and concrete specimen temperatures

variation of 28-day tests on standard-cured specimens for each type of cement averaged about 5 percent, which indicates good control of the testing procedure. Considerably better control was obtained with Types I and II cements than with Type V cement as two brands of Type V cement were used without blending.

Series 1

At very early ages the strength of the immediately and continuously steam-cured concrete specimens increased with the temperature to 165 F (see Fig. 2). Temperatures of 165 F and higher produced equal strengths at age 6 hours, but at later ages specimens cured above 165 F gained strength more slowly than specimens cured at the lower temperatures. When steam curing was continued for 24 hours, there was no appreciable difference in strength between concretes cured at 130 and 165 F, and these temperatures gave higher strengths than either higher or lower temperatures. This same relationship was also true for steam curing for 2 and 3 days; *i.e.*, temperatures between 130 and 165 F produced maximum strengths. After 48 hours, specimens continuously steam cured at temperatures of 185 and 195 F had lower strengths than cylinders continuously fog cured for the same length of time. Fig. 2

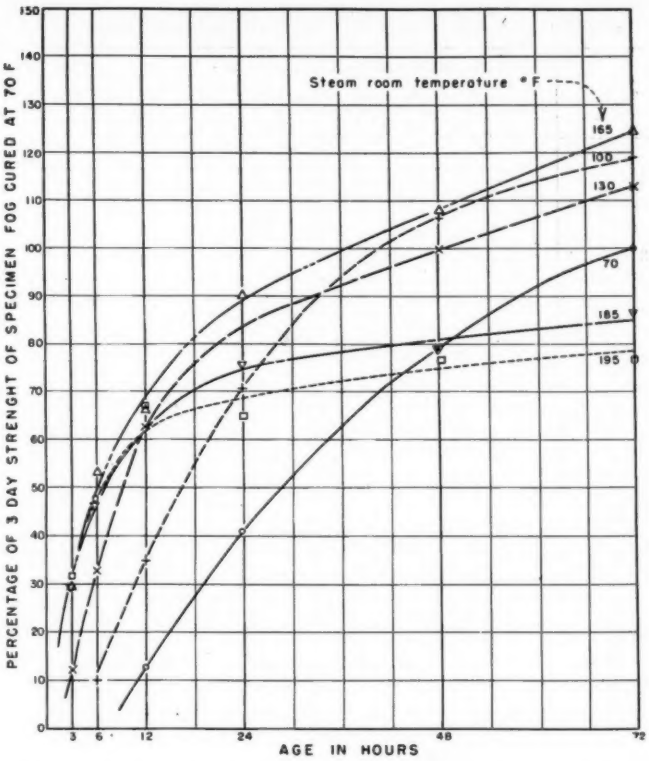

Fig. 2—Effect of steam curing at temperatures below 200 F on the compressive strength of concrete at early ages, Type I cement

Steam curing started immediately after specimens were cast. Compressive strength at 3 days of specimens fog cured at 70 F was 2250 psi.

shows the strength of steam-cured specimens expressed as a percentage of the 3-day strength of a specimen fog cured at 70 F. This figure is for a Type II cement but Types I and V cements produced similar results. Data for these tests are given in Table 2.

Severe swelling and some cracking around the circumference, near the ends of the cylinders, was noted when specimens were cured at 185 F and higher. This distress had been noted[2] in the manufacture of concrete masonry units when the temperature of a freshly molded block was raised too rapidly.

TABLE 2—COMPRESSIVE STRENGTH OF 6 x 12-IN. CYLINDERS—PLATTE RIVER AGGREGATE—FIRST SERIES

Time cured	Compressive strength (psi)†								
	Fog-cured control cylinders 70 F			Steam cured at 100 F			Steam cured at 130 F		
	Type I	Type II	Type V	Type I	Type II	Type V	Type I	Type II	Type V
3 hr	—	—	—	—	—	—	280	180	40
6 hr	—	—	—	190	140	110	780	560	280
12 hr	280§	280	140	640	640	510	1520	1120	720
1 day	920	850	500	1300	1320	1060	2020	1590	1220
2 days	1780	1510	1070	1960	1880	1510	2400	1930	1830
3 days	2250	1980	1500	2180	2180	1830	2720	2360	2100
4 days	—	—	—	2300	2490	1980	—	—	—
7 days	3170	2940	2350	2670	2890	2390	—	—	—
28 days	4680	4740	4060	—	—	—	—	—	—
	Steam cured at 165 F			Steam cured at 185 F			Steam cured at 195 F		
	Type I	Type II	Type V	Type I	Type II	Type V	Type I	Type II	Type V
3 hr	670	420	120	660	560	360	780	590	90
6 hr‡	1220	860	490	1040	820	660	1160	880	340
12 hr	1750	1310	1000	1420	1280	1050	1640	1080	620
1 day	2070	1600	1340	1710	1500	1240	1580	1400	610
2 days	2480	2010	1780	1800	1730	1500	1880	1560	700
3 days	2880	2240	1960	1760	1740	1580	1880	1550	860
7 days	2750	2300	2030	1960	1790	1810	1790	1620	840

†Average of two cylinders.
‡185 F specimens steam cured 5 hours.
§Average of 10 cylinders.

Series 2

Data obtained from this series of tests, Table 3, amplify the previous test and show that steam curing at temperatures of 100 and 130 F followed by fog curing did not greatly reduce the strength of the concrete at 28 days (see Fig. 3). Any amount of steam curing for periods between 6 hours and 3 days at 165 F definitely limited the strength at 28 days to less than 75 percent of that of specimens continuously fog cured at 70 F, and a temperature of 185 F limited the 28-day strength to about one-half that obtained on similar specimens fog cured at 70 F (see Fig. 4). Concrete steam cured at 185 F for 3 days showed no appreciable gain in strength during the fog-curing period that followed. In all cases the continuously fog-cured specimens had higher strengths at 28 days than specimens subjected to any of the various steam-curing conditions, except specimens containing Type V cement steam cured at low temperatures for 1 day. These data in Table 3 also show that, in general, the longer the concrete was subjected to steam curing, the lower the 28-day strength. The advantage of continuous steam curing decreases rapidly with age, and a comparison of Curves 2 and 3 on Fig. 3 shows that after 24 hours of steam curing, moist curing at 70 F is only slightly less effective than an equal period of continued steam curing.

Series 3

Data for these tests are given in Table 4 and Fig. 5, and show that a delay of a few hours prior to steam curing produced higher strengths at

TABLE 3—COMPRESSIVE STRENGTH OF 6 x 12-IN. STEAM CURED CYLINDERS—PLATTE RIVER AGGREGATE—SECOND SERIES

Time specimens subjected to steam and fog curing	Age at test	Compressive strength (psi)*									
		100 F†			130 F			165 F		185 F	
		Type I	Type II	Type V	Type I	Type II	Type V	Type I	Type II	Type I	Type II
6-hour steam cure	6 hours	130	120	140	280	400	390	620	740	880	1010
1-day steam cure	1 day	1090	1280	1210	1400	1620	1430	1550	1990	1420	1660
3-day steam cure	3 days	2010	2460	2220	2620	2780	2530	2450	2840	2000	2160
6-hour steam cure plus 7-day fog cure	7 days	2460	2770	2680	2470	2730	2390	2200	2210	1720	1680
6-hour steam cure plus 28-day fog cure	28 days	3510	4640	4200	3920	4490	4000	2820	3380	2220	2310
1-day steam cure plus 6-day fog cure	7 days	2550	2960	2450	2700	3220	2680	2700	2950	1670	1670
1-day steam cure plus 27-day fog cure	28 days	3760	4450	4220	3750	4270	3830	3200	3370	2080	2340
3-day steam cure plus 4-day fog cure	7 days	2660	2960	2580	2970	3330	3050	2830	2930	2320	2180
3-day steam cure plus 25-day fog cure	28 days	3460	4260	4030	3710	4250	3980	2800	3550	1920	2290

Fog-cured control cylinders 70 F

Time cured	Age	Compressive strength (psi)		
		Type I‡	Type II‡	Type V§
1-day	1 day	540	680	600
3-day	3 days	1680	1890	1560
7-day	7 days	2640	3010	2350
28-day	28 days	4070	4810	4100

*Average of two cylinders.
†Steam room temperature.
‡Average of eight cylinders.
§Average of four cylinders.

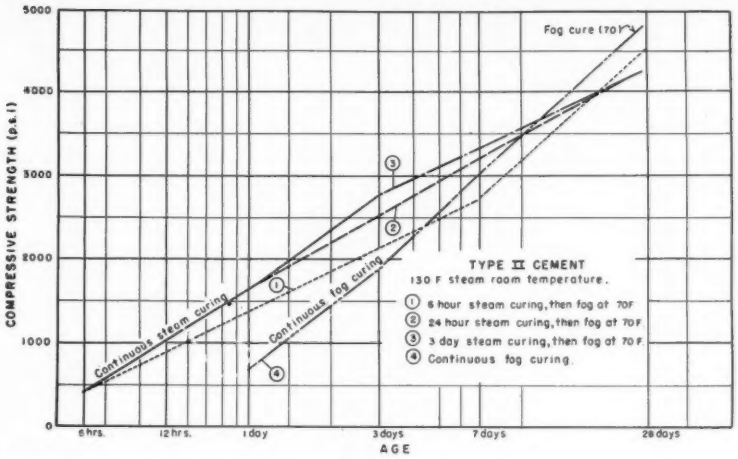

Fig. 3—Concrete given preliminary steam curing at 130 F continues to gain strength during storage in fog room

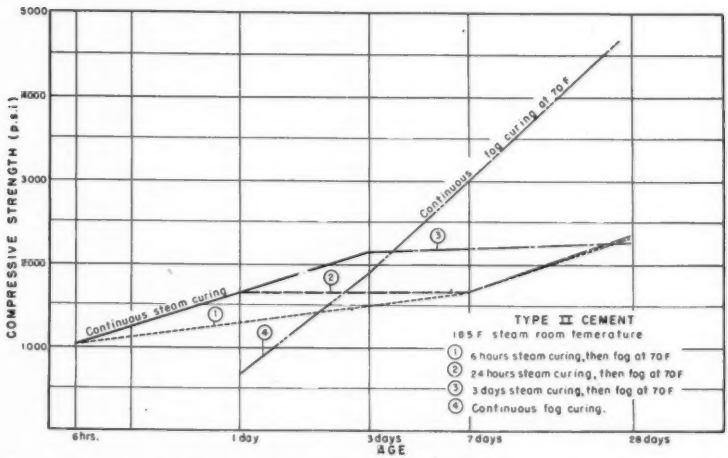

Fig. 4—Concrete given preliminary steam curing at 185 F shows little gain in strength during subsequent fog room storage

TABLE 4—COMPRESSIVE STRENGTH OF 6 x 12-IN. STEAM CURED CYLINDERS —PLATTE RIVER AGGREGATE—THIRD SERIES

Specimens fog cured for period indicated, then steam cured to age of 24 hours	Compressive strength (psi) at age of 1 day							
	100 F (steam temp)		130 F (steam temp)		165 F (steam temp)		185 F (steam temp)	
	Type I	Type II	Type I	Type II	Type I	Type II	Type I	Type II
0 hours	1070*	1190	1360	1500	1520	1600	1350	1630
½ hour	1060	1200	1400	1600	1680	1640	1310	1380
1 hour	1150	1260	1320	1600	1660	1500	1380	1360
2 hours	1280	1390	1520	1860	1600	1740	1280	1600
4 hours	1190	1200	1720	1830	1950	1990	1970	2060
6 hours	1120	1120	1540	1720	1900	2050	2020	2260
8 hours	1050	1070	1550	1660	1850	1950	1900	2160
12 hours	940	980	1420	1480	1710	1830	1880	2040

Fog-cured control cylinders 70 F

Continuously fog-cured at 100 percent R.H. 70 F	Compressive strength (psi)	
	Type I	Type II
1 day	520†	680
2 days	1220	1300
7 days	2760	2800

*Average of two cylinders.
†Average of eight cylinders.

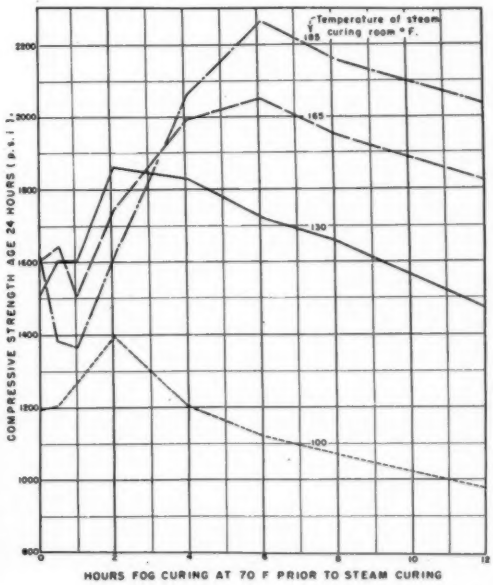

Fig. 5—Higher temperatures require a longer delay prior to steam curing to provide maximum strengths at 24 hours, Type II cements

Specimens were fog cured for various initial periods, then steam cured to age 24 hours and tested.

24 hours than if the specimens were subjected to steam curing immediately after casting. Higher steam-curing temperatures required longer fog-curing periods prior to steam curing to produce maximum strengths. The highest strengths at age 24 hours, at a temperature of 185 F, were obtained with about a 6-hour delay prior to steam curing. A 2-hour delay for steam curing at 100 F was the optimum, and at 130 and 165 F a delay of about 4 hours produced maximum strengths. Higher temperatures produced higher strengths for the optimum delay period.

CONCLUSIONS AND RECOMMENDATIONS

These tests cover only a small portion of the possible field of investigation on the subject of steam curing and it is recommended that additional tests be made to study the effect of steam curing on durability, volume changes, elastic properties, and strength at later ages of concrete. A more thorough study should be made involving delayed curing, longer curing periods, and various rates of heating and cooling. However, from the tests already completed the following conclusions can be made:

1. Steam curing accelerated the early strength development of the concrete. However, any amount of steam curing, regardless of temperature, with the exception of specimens containing Type V cement steam cured at low temperatures for 1 day, had an adverse effect upon the compressive strength at 28 days compared to specimens continuously fog cured at 70 F.

2. Optimum steam-curing temperatures were between 130 and 165 F, and about 8 hours steam curing in this range produced strengths equal to the 24-hour strength of concrete fog cured at 70 F.

3. Specimens steam cured at temperatures of 100 and 130 F for periods up to 3 days and then stored in the fog room continued to gain strength under moist curing and had only slightly less strength at 28 days than continuously fog-cured specimens.

4. Specimens steam cured at a temperature of 165 F for periods up to 3 days showed little gain in strength during subsequent fog curing and at 28 days had compressive strengths about 75 percent of continuously fog-cured specimens.

5. A delay of 2 to 6 hours prior to steam curing, depending upon the temperature, produced strengths 15 to 40 percent higher at age 24 hours than when steam curing was started immediately after the concrete was placed.

6. Temperatures above 165 F and rapid temperature rise were detrimental to the strength of concrete.

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Charts are presented which give direct values of np and $f_s \div nf_c$ when structural members are subject to bending and direct compression.

A Shortcut for Determining Reinforcement in Reinforced Concrete*

By V. BOGVAD-CHRISTENSEN†

SYNOPSIS

This paper presents a graph which gives the complete relationship between moments, thrusts, concrete dimensions, reinforcement and resulting stresses for the reinforced concrete members in question.

INTRODUCTION

The graph derived for bending and direct stress in reinforced concrete consists of three parts: (1) reinforcement in tension face only where $\Delta = 0$, (2) reinforcement in tension face and a fractional amount of reinforcement in the compression face where $0 < \Delta < 1$, and (3) same amount of reinforcement in tension and compression face where $\Delta = 1$.

This graph is particularly useful in the design or analysis of columns, arches, dams, etc. The reinforcement is obtained directly, when moments, thrusts, dimensions and permissible stresses are given.

The writer became acquainted with a similar graph in the office of Christiani and Nielsen, Engineers and Contractors, Copenhagen, Denmark, 35 years ago, but recalls neither the name of the author or details of the graph. The graph was—as far as he knows—never published.

DERIVATION

The fundamental formulas for the relationship between moments, thrusts, dimensions, reinforcement, and stresses are given below.

M = moment
N = thrust
b = width
h = height
k = ratio of depth of neutral axis to total height
f_c = unit stress in concrete at extreme fiber

*Manuscript received by the Institute, Aug. 22, 1949. Title No. 46-19 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 4, Dec. 1949, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Apr. 1, 1950. Address 7400 Second Boulevard, Detroit 2, Mich.

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f_s = unit stress in reinforcement in tension
f'_s = unit stress in reinforcement in compression
a = distance between center of bar and center of section to total height
p = ratio of compression reinforcement to tension reinforcement
Δ = ratio of compression reinforcement to concrete section
n = ratio of modulus of elasticity of steel to modulus of elasticity of concrete

Taking ΣM about N yields:

$$\tfrac{1}{2}k\left(\tfrac{1}{2}-\frac{k}{3}\right)+pa\left(\Delta\frac{f'_s}{f_c}+\frac{f_s}{f_c}\right)=\frac{M}{f_c bh^2} \quad \text{(Unit used for graph)} \quad (1)$$

Summing up forces yields:

$$\tfrac{1}{2}k+p\left(\Delta\frac{f'_s}{f_s}-\frac{f_s}{f_c}\right)=\frac{N}{f_c bh} \quad \text{(Unit used for graph)} \quad (2)$$

Ratio of $\frac{f_s}{n}$ to f_c in terms of a and k

$$\frac{1+2a}{2k}-1=\frac{f_s}{n f_c} \quad \text{(Unit used for graph)} \quad (3)$$

Ratio of $\frac{f'_s}{n}$ to f_c in terms of a and k:

$$1-\frac{1-2a}{2k}=\frac{f'_s}{n f_c} \quad (4)$$

To get these equations in more convenient form the values $\frac{f_s}{f_c}$ and $\frac{f'_s}{f_c}$ from Eq. (3) and (4) are inserted in Eq. (1) and (2):

$$\tfrac{1}{2}k\left(\tfrac{1}{2}-\frac{k}{3}\right)+\frac{npa}{2k}\left[\Delta\,(2a+2k-1)+(2a-(2k-1))\right]=\frac{M}{f_c bh^2} \quad (5)$$

$$\tfrac{1}{2}k+\frac{np}{2k}\left[\Delta\,(2a+2k-1)-(2a-(2k-1))\right]=\frac{N}{f_c bh} \quad (6)$$

$$\frac{2a-(2k-1)}{2k}=\frac{f_s}{n f_c} \quad (7)$$

$$\frac{2a+2k-1}{2k}=\frac{f'_s}{n f_c} \quad (8)$$

By substituting x for $2k-1$ the fundamental equations are:

$$\frac{(x+1)(2-x)}{24}+\frac{npa}{x+1}\left[\Delta\,(2a+x)+(2a-x)\right]=\frac{M}{f_c bh^2} \quad (9)$$

$$\frac{x+1}{4}+\frac{np}{x+1}\left[\Delta\,(2a+x)-(2a-x)\right]=\frac{N}{f_c bh} \quad (10)$$

$$\frac{2a-x}{x+1}=\frac{f_s}{n f_c} \quad (11)$$

$$\frac{2a+x}{x+1}=\frac{f'_s}{n f_c} \quad (12)$$

WHEN $\Delta = 0$

For this condition Eq. (9), (10), (11) and (12) are reduced to:

$$\frac{(x+1)(2-x)}{24} + \frac{npa}{x+1}(2a-x) = \frac{M}{f_c bh^2} \quad \ldots\ldots(13)$$

$$\frac{x+1}{4} - \frac{np}{x+1}(2a-x) = \frac{N}{f_c bh} \quad \ldots\ldots(14)$$

$$\frac{2a-x}{x+1} = \frac{f_s}{nf_c} \quad \ldots\ldots(15)$$

$$\frac{2a+x}{x+1} = \frac{f'_s}{nf_c} \quad \ldots\ldots(16)$$

By inserting different values for x and np ($a = 0.4$) in Eq. (13) and (14) a table was prepared for the values $\frac{M}{f_c bh^2}$ and $\frac{N}{f_c bh}$ and a second table for the values $\frac{f_s}{nf_c}$ and $\frac{f'_s}{nf_c}$ was derived from Eq. (15) and (16).

From these two tables the zone $\Delta = 0$ was constructed. This part of the graph is enlarged and presented separately (Fig. 1).

WHEN $0 < \Delta < 1$

It is obviously the designers aim to use a minimum of reinforcement. The quantity $np\,(1+\Delta)$ is therefore derived as a minimum by differentiation.

An equation for the value $np\,(1+\Delta)$ was obtained from Eq. (9) and (10) by first deriving from these equations:

$$np\,\Delta = \frac{x+1}{4a(2a+x)}\left[2\frac{M}{f_c bh^2} + 2a\frac{N}{f_c bh} - \frac{x+1}{4}\left(2a + \frac{2-x}{3}\right)\right] \quad \ldots\ldots(17)$$

$$np = \frac{x+1}{4a(2a-x)}\left[2\frac{M}{f_c bh^2} - 2a\frac{N}{f_c bh} + \frac{x+1}{4}\left(2a - \frac{2-x}{3}\right)\right] \quad \ldots\ldots(18)$$

and then adding these two

$$np\,(1+\Delta) = \frac{x+1}{4a^2 - x^2}\left[2\frac{M}{f_c bh^2} - \frac{N}{f_c bh}x + \frac{x^2}{3} - \frac{1-x}{6}\right] \quad \ldots\ldots(19)$$

By differentiation:

$$\frac{d\,[np\,(1+\Delta)\,]}{dx} = \frac{4a^2\left(2\frac{M}{f_c bh^2} - \frac{N}{f_c bh}\right) + x\left[4a^2 - \frac{1}{3} + 4\frac{M}{f_c bh^2} - 2\frac{N}{f_c bh}4a^2\right]}{(4a^2 - x^2)^2} +$$

$$\frac{x^2\left[2\frac{M}{f_c bh^2} - \frac{N}{f_c bh} + 4a^2\right] - \frac{x^4}{3}}{(4a^2 - x^2)^2} = 0$$

Since $(4a^2 - x^2)^2 > 0$ the numerator $= 0$.

It may be more conveniently written in the form:

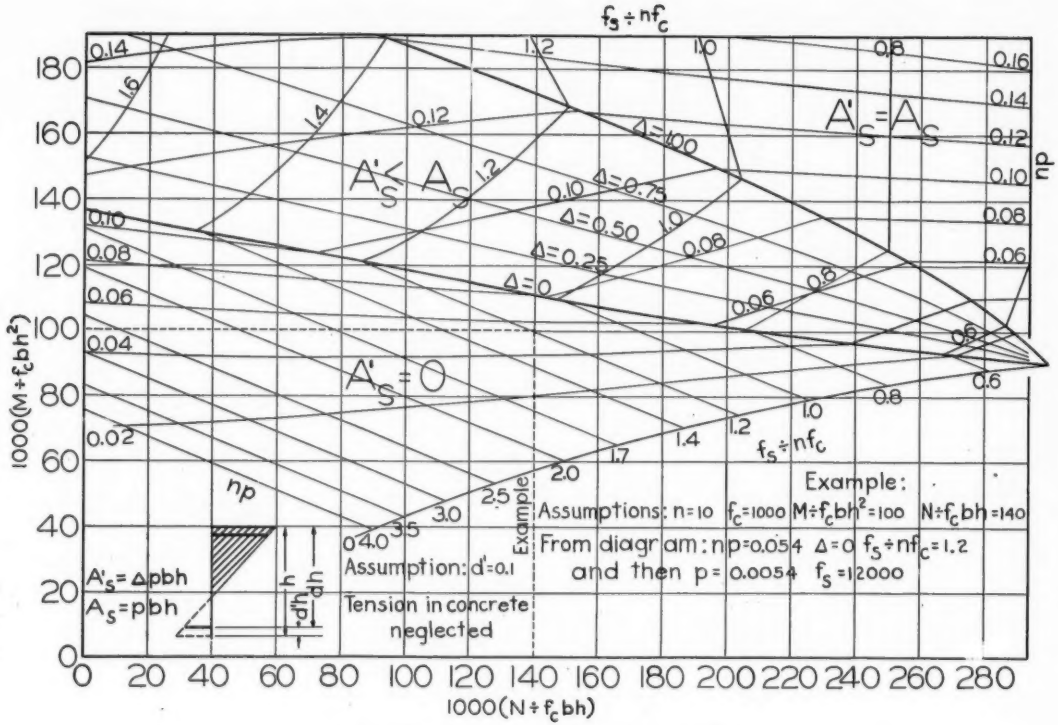


Fig. 1—The $\Delta = 0$ zone of the graph, $d' = 0.10$

$$\frac{M}{f_c\,bh^2} = \frac{4a^2\,\frac{N}{f_c\,bh} - x\left[4a^2 - \tfrac{1}{3} - 2\,\frac{N}{f_c\,bh}\,4a^2\right] - x^2\left[4a^2 - \frac{N}{f_c\,bh}\right]}{2\,(4a^2 + 2\,x + x^2)} \quad \ldots\ldots (20)$$

By inserting values of x and $\frac{N}{f_c bh}$ in Eq. (20) a table was prepared for the value $\frac{M}{f_c bh^2}$.

Inserting values of $\frac{M}{f_c bh^2}$ and $\frac{N}{f_c bh}$ with the x value obtained from the above mentioned table into Eq. (17) and (18) a table was obtained for $np\,\Delta$ and np.

From Eq. (15) and (16), Eq. (20), and Eq. (17) and (18) this zone of the graph was constructed (Fig. 2).

WHEN $\Delta = 1$

$$\frac{(x+1)\,(2-x)}{24} + 4a^2\,\frac{np}{x+1} = \frac{M}{f_c\,bh^2} \quad \ldots\ldots (21)$$

$$\frac{x+1}{4} + \frac{2\,np\,x}{x+1} = \frac{N}{f_c\,bh} \quad \ldots\ldots (22)$$

$$\frac{2\,a - x}{x+1} = \frac{f_s}{n\,f_c} \quad \ldots\ldots (23)$$

$$\frac{2\,a + x}{x+1} = \frac{f'_s}{n\,f_c} \quad \ldots\ldots (24)$$

From Eq. (21) and (22) a table for the values $\frac{M}{f_c bh^2}$ and $\frac{N}{f_c bh}$ was computed by inserting values for np and x. From this table and the table from Eq. (15) and (16) this part of the graph was constructed (Fig. 2).

EXAMPLES

Derivation of reinforcement

Example 1

Assumptions: $n = 10$; $f_c = 1000$; $\frac{M}{f_c bh^2} = 0.147$; $\frac{N}{f_c bh} = 0.150$

From graph (Fig. 2): $np = 0.105$ $\quad \Delta = 0.75$ $\quad f_s \div nf_c = 1.14$

$p = 0.0105$ $\quad \Delta p = 0.0078$ $\quad f_s = 11{,}400$

Example 2

Assumptions: $n = 10$; $f_c = 1200$; $\frac{M}{f_c bh^2} = 0.080$; $\frac{N}{f_c bh} = 0.141$

From graph (Fig. 2): $np = 0.02$ $\quad \Delta = 0$ $\quad f_s \div nf_c = 1.6$

$p = 0.002$ $\quad f_s = 19{,}200$

If permissible $f_s = 18{,}000$ it indicates that the dimensions assumed are too small to use $f_c = 1200$. The dimensions should either be in-

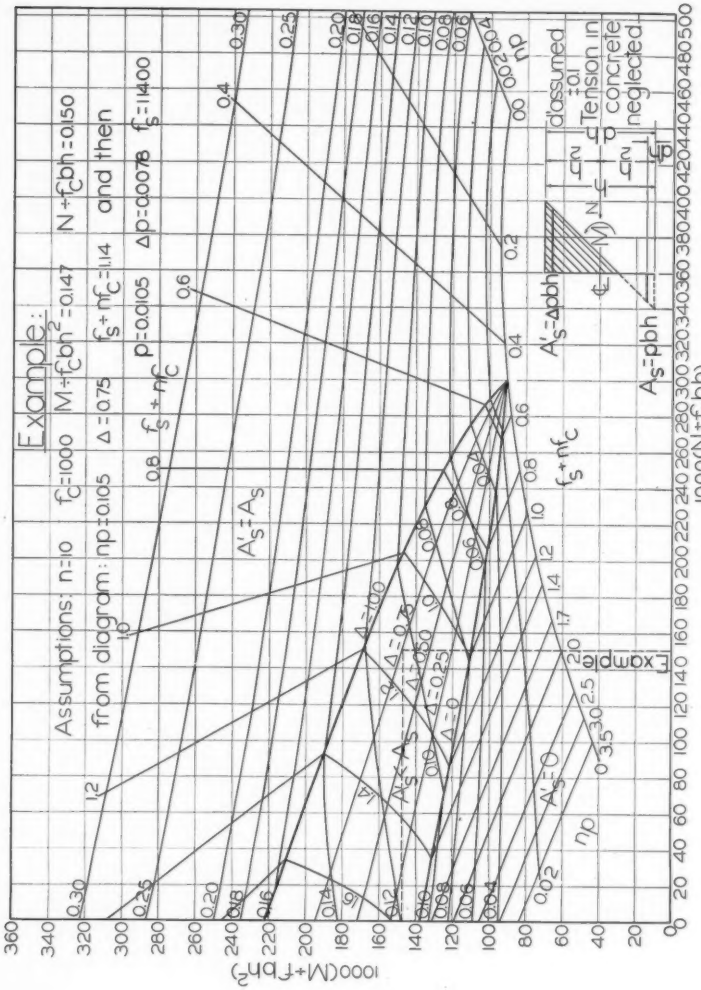


Fig. 2—Chart for determining reinforcement, d' = 0.10

creased or the graph entered with a smaller value of f_c which naturally would increase the reinforcement.

Example 3

Assumptions: $n = 10$; $f_c = 1250$; $\frac{M}{f_c\, bh^2} = 0.240$; $\frac{N}{f_c\, bh} = 0.054$

From graph (Fig. 2): $np = 0.20$ $\quad \Delta = 1$ $\quad f_s \div nf_c = 1.4$

$p = 0.02$ $\quad f_s = 17{,}500$

Analysis of stresses

Example 4

Assumptions: $n = 10$; $\frac{M}{bh^2} = 114$; $\frac{N}{bh} = 76$; $\frac{\frac{M}{f_c\, bh^2}}{\frac{N}{f_c\, bh}} = \frac{114}{76} = 1.5$ (draw this line on graph Fig. 1)

$np = 0.08$

From graph (Fig. 1): $\frac{M}{f_c\, bh^2} = 0.114$ $\quad f_c = \frac{114}{0.114} = 1000$

$f_s \div n f_c = 1.4$ $\quad f_s = 14{,}000$

Although the value $d' = 0.1$ represents the most common value, it is relatively easy—in most cases—to adjust the obtained results approximately to other values of d'. The writer hopes later to prepare similar graphs for the values $d' = 0.05$ and $d' = 0.15$.

Discussion of a paper by V. Bogvad-Christensen:

A Shortcut for Determining Reinforcement in Reinforced Concrete*

By WALTER LUM and AUTHOR

By WALTER LUM†

Much has been written about combined bending with axial load for rectangular concrete sections. Because of the few initial conditions of the problem, the final formulas presented by many writers are usually the same except for the arrangement of terms. The author, however, introduces in his final arrangement an $M/f_c bh^2$ term, which is similar to Bakhoum's M_s/bd^2f_c.‡ Both authors plot graphs involving these terms to simplify computation.

Of how much use are these charts? The draftsmen in a design office are often accused of being narrow minded individuals when it comes to accepting formulas, theories, or supposed shortcuts. Do you blame them if simpler solutions already exist? The usual problem of this type is presented in the form of a given concrete section subject to a bending moment M and an axial load N. The author presents his first example as $M/f_c bh^2 = 0.147$ and $N/f_c bh = 0.150$. Without the use of the author's curves, it is impossible to picture the original conditions of the problem. What is M? What is N?

Moreover, the general practitioner is reluctant to use any chart he can not verify himself without too much effort. For the many variables involved in most problems, a chart is somewhat limited in scope, although it is often put to good advantage, usually only by its maker. Formulas may be developed for the general case, but to be of use they must be simple. Nevertheless, since the Portland Cement Association's two-cycle moment distribution analysis is presently in vogue, combined bending with axial load is forever present, and structural sections must be designed accordingly. The structural draftsman today designs members subject to bending with direct stress as readily as simple beams. In the balance of this discussion an attempt will be made to illustrate what the writer considers the most direct procedure. The

*ACI JOURNAL, Dec. 1949, *Proc.* V. 46, p. 285. Disc. 46-19 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

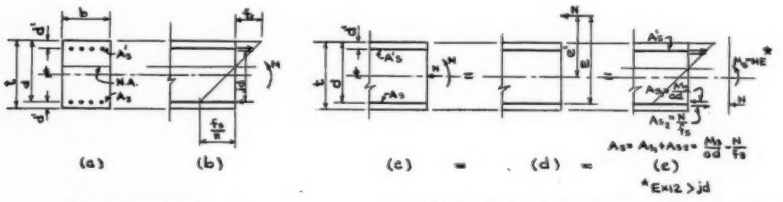
†Honolulu, Hawaii.

‡Bakhoum, M., "Direct Dimensioning of Rectangular Sections," ACI JOURNAL, Jan. 1949, *Proc.* V. 45, p. 381.

procedure used below is not new. It has been suggested by several writers,* but has yet to find itself into the vernacular of text book authors.

SUGGESTED PROCEDURE

There is no fundamental difference in the design of flexural members for simple bending and for those cases of bending with axial load in which tension exists over a considerable part of the section. Perhaps the most rapid solution is to choose a section as one would for a beam, convert the problem to an equivalent moment and axial load on the tensile steel, and to proportion the section to the requirements of the equivalent moment and axial load. Fig. A (c), (d) and (e) is a pictorial derivation of the fundamental formulas. In fact, all of the essential design formulas for flexural members are summed up in Fig. A. It is assumed the reader is familiar with the ACI *Reinforced Concrete Design Handbook* and the nomenclature used here is obvious.



SIMPLE BENDING		COMBINED BENDING WITH AXIAL LOAD	
Bal. design	Comp. steel design	Bal. design	Comp. steel design
$d = \sqrt{\frac{12000\,M}{Kb}}$	d = fixed	$d^2 - \frac{1000\,Nd}{2Kb} = \frac{1000\,N\,(2e' - d')}{2Kb}$	d = fixed
$A'_s = 0$	$A'_s = \frac{M - KF}{cd}$	$A'_s = 0$	$A'_s = \frac{M_s - KF}{cd}$
$A_s = \frac{M}{ad}$	$A_s = \frac{M}{ad}$	$A_s = \frac{M_s}{ad}$	$A_s = \frac{M_s}{ad} - \frac{N}{f_s}$ (comp. thrust)
			$A_s = \frac{M_s}{ad} + \frac{N}{f_s}$ (ten. thrust)

Fig. A—Design formulas for flexure—For a flexural member subject to an axial load, an arbitrary procedure is to replace the equivalent eccentric load by an equal load acting on the tension steel and a moment ($M_s = Ne$)

The sample calculations (Fig. B) illustrate the routine design procedure for flexural members. The value KF equals $Kbd^2/12{,}000$, the resisting moment of the concrete; a equals $f_s j/12{,}000$, a constant for a given unit steel stress and an average j value;† c is a constant similar to a and is adequately derived and tabulated in the ACI handbook. The formula for the depth required for a balanced design without compressive reinforcement is somewhat different.

*See bibliography.

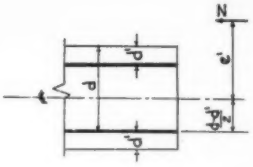
†The value a is based upon a j for beams reinforced with tension steel only, the value of A_s computed from using a is approximate but sufficiently accurate for most cases.

It is derived from $d = \sqrt{\dfrac{M_s\ 12{,}000}{Kb}}$

where $M_s \times 12{,}000 = 1000\ N\left(e' + \dfrac{d - d'}{2}\right)$

and $d = \sqrt{\dfrac{1000\ N\left(e' + \dfrac{d - d'}{2}\right)}{Kb}}$

from which $d^2 - \dfrac{1000\ Nd^2}{2Kb} = \dfrac{1000\ N(2e' - d')}{2Kb}$



MINIMUM STEEL

It is also interesting to note that the use of a higher steel stress in a beam where compressive reinforcement is required does not necessitate a more economical design. In the example illustrated in the sample calculation (Fig. B) for bending with axial load where compressive reinforcement is required, a total steel area of 4.86 sq in. is required for a 20,000 psi tensile steel stress. The neutral plane is kd or 8.07 in. from the top. What happens if the neutral plane is lowered, say to 9.19 in. from the top by using a 16,000 psi steel stress? The total steel area required is 4.67 sq in. By comparing the two solutions, the lower unit steel stress requires 4 percent less steel than the 25 percent higher unit steel stress.

This is not a paradox. The steel area required is less in the second case, but the area of the tensile steel is greater. Increase of tensile steel pulls the neutral plane down and thereby produces a greater resisting moment in the concrete. Actually the steel does more good in the tensile side of the section and therefore a lower steel stress is preferable.

Solving the same problem by two of the three fundamental static equations, it is evident that the neutral plane may be moved to any desired position as long as the areas of tensile and compressive steel are adjusted so that the total tension on a section balances the total compression. Location of the neutral plane for a minimum amount of steel may be determined by calculus or a few trial sections. Steel areas for the above problem were computed for 10,000, 14,000, 16,000, 18,000 and 20,000 psi and plotted in Fig. C. The curves in Fig. C show that when the design steel stress is increased from 10,000 to 20,000 psi A_s decreases and A'_s increases, but the total $A_s + A'_s$ changes relatively little. The minimum steel area is practically constant between 13,000 and 18,000 psi, which means that the most economical steel stress need not be determined with any great degree of accuracy. For e'/t less than 2, 16,000 psi is a good average value. A higher steel stress should be used for e'/t greater than 2.

The compressive steel required may be further reduced by taking advantage of Section 706(*b*) of the ACI Building Code which states, "The effectiveness of compression reinforcement in resisting bending may be taken at twice the value indicated from the calculations assuming a straight line relation between stress and strain and the modular ratio given in Section 601, but not greater than the allowable stress in tension."

SIMPLE BENDING—BALANCED DESIGN

Given: $M = 94.4$, $b = 12$

Design constants: 20,000/10/1350; $K = 236$, $a = 1.44$

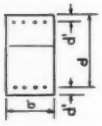
Beam section:

$$d = \sqrt{\frac{12{,}000\,M}{Kb}} = \sqrt{\frac{12{,}000 \times 94.4}{236 \times 12}} = 20 \text{ in.}$$

$$A_s = \frac{M}{ad} = \frac{94.4}{1.44 \times 20} = 3.28 \text{ sq in.}$$

SIMPLE BENDING—COMPRESSIVE STEEL REQUIRED

Given: $M = 133.2$, $b = 12$, $d = 20$, $d' = 2$



Design constants: 20,000/10/1350; $K = 236$, $a = 1.44$, $F = 0.400\,KF = 94.4$, $\frac{d'}{d} = 0.10$, $c = 0.69$ (Table 7, ACI Handbook)

Reinforcing required:

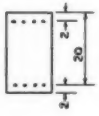
$$A'_s = \frac{M - KF}{cd} = \frac{133.2 - 94.4}{0.69 \times 20} = 2.81 \text{ sq in.}$$

$$A_s = \frac{M}{ad} = \frac{133.2}{1.44 \times 20} = 4.63 \text{ sq in.}$$

COMBINED BENDING WITH AXIAL LOAD—COMPRESSIVE STEEL REQUIRED

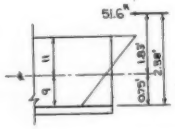
Given: $M = 94.4$, $N = 51.6$, $b = 12$

Assume:



Design constants: 20,000/10/1350; $K = 236$, $a = 1.44$, $F = 0.400\,KF = 94.4$, $\frac{d'}{d} = 0.10$, $c = 0.69$

M_s = NE



Reinforcing required:

$$A'_s = \frac{M_s - KF}{cd} = \frac{133.2 - 94.4}{0.69 \times 20} = 2.81 \text{ sq in.}$$

$$A_s = \frac{M_s}{ad} - \frac{N}{f_s} = \frac{133.2}{1.44 \times 20} - \frac{51.6}{20} = 2.05 \text{ sq in.}$$

$M_s = 51.6 \times 2.58 = 133.2$

Total steel $A'_s + A_s = 4.86$ sq in.

Alternate design:

16,000/10/1350 $M_s = 133.2$

$K = 262$, $a = 1.13$

$F = 0.400\,KF = 104.8$

$\frac{d'}{d} = 0.10$, $c = 0.71$

$$A'_s = \frac{133.2 - 104.8}{0.71 \times 20} = 2.00 \text{ sq in.}$$

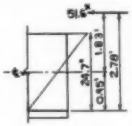
$$A_s = \frac{133.2}{1.13 \times 20} - \frac{51.6}{16} = 2.67 \text{ sq in.}$$

Total steel $A'_s + A_s = 4.67$ sq in.

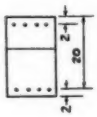
Note: M_s is an equivalent moment about the tensile steel and can be readily estimated since a rough depth must be assumed before analysis can begin on a practical structure. When compressive steel is required, a lower tensile steel stress usually requires less steel area than $f_s = 20{,}000$ psi.

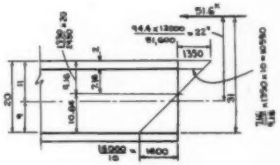
Fig. B—Sample calculations

COMBINED BENDING WITH AXIAL LOAD—BALANCED DESIGN

Given	Design constants	Depth required	M_o = NE	Steel req'd
M = 94.4	20,000/10/1350	$e' = \frac{M \times 12{,}000}{N \times 1000}$		$A_s = \frac{M_o}{ad} - \frac{N}{f_s}$
N = 51.6	K = 236 a = 1.44	$= \frac{94.4 \times 12}{51.6} = 22$		$= \frac{143.5}{1.44 \times 24.7} - \frac{51.6}{20}$
b = 12		$d^2 = \frac{1000\,Nd}{2Kb}$	$M_o = 51.6 \times 2.78 = 143.5$	
d' = 2		$= \frac{1000\,N(2e' - d')}{2Kb}$		$A_s = 1.45$ sq in.
		$d^2 = \frac{51{,}600\,d}{2 \times 236 \times 12}$		
		$= \frac{51{,}600\,(2 \times 22 - 2)}{2 \times 236 \times 12}$		
		d = 24.7		

SOLUTION for COMBINED BENDING WITH AXIAL LOAD (Without the use of formulas)

Given	Assume	Design specs.
M = 94.4 ft-kips		16,000/10/1350
N = 51.6 kips		
b = 12 in.		



Compressive steel required:

Bal. moment about ten. steel, $M_o = 51{,}600 \times 31 = 1{,}600{,}000$ in.-lb

Concrete compression $= \frac{1350 \times 12 \times 9.16}{2} = 74{,}200$ lb

Concrete resisting moment $= 74{,}200\,(10.84 + 2/3 \times 9.16) = 1{,}255{,}000$ in.-lb

Comp. steel resisting moment $= 345{,}000$ in.-lb

Steel compression $= \frac{345{,}000}{18} = 19{,}200$ lb.

Comp. steel req'd $= \frac{19{,}200}{10{,}550} \times \frac{10}{9} = 2.02$ sq in.

$$\frac{10}{9} = \frac{n}{n-1}$$

Tensile steel required:

Tension = compression

Concrete compression	= 74,200 lb
Steel compression	= 19,200 lb
Total compression	= 93,400 lb
Axial load	= 51,600 lb
Total tension	= 41,800 lb

Ten. steel req'd $= \frac{41{,}800}{16{,}000} = 2.62$ sq in.

Fig. B (cont'd)—Sample calculations

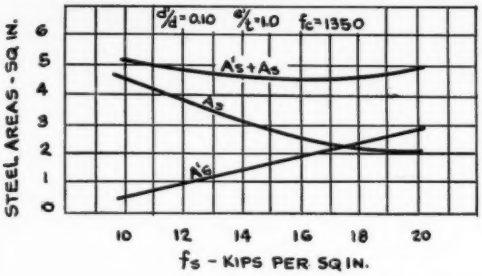


Fig. C—Minimum steel area ($A'_s + A_s$): As f_s increases, A_s decreases but A'_s increases. The total steel area is fairly constant for a wide range of values of f_s. 16,000 psi is an expedient value.

BENDING IN COLUMNS

So far the problem of combined bending and direct stress has been clear cut. The actual behavior of columns subject to bending is far from the assumed straight line stress-strain relationship for flexural members. A different and more practical treatment is presented in the Portland Cement Association's Structural Information Sheet ST 64 "Combined Bending and Axial Load."

Section 2 of the publication points out some of the inconsistencies of the present ACI Code. Three design conditions are considered and analyzed in connection with a rectangular section with symmetrical tied reinforcement where $e'/t = 1$ with the following results:

(a) Concrete uncracked, allowable stress f_p, $N = 76$ kips
(b) Concrete cracked, allowable stress f_c, $N = 61$ kips
(c) Concrete cracked, allowable stress f_p, $N = 51$ kips

In accordance with the Code provisions, the reinforced concrete section is capable of sustaining $N = 76$ kips for a value of e'/t slightly less than 1 and 51 kips for a value of e'/t slightly greater than 1. Using a beam design, 61 kips is allowed. Evidently the variation in results is undesirable. For a more complete discussion of this problem the reader is referred to the PCA publication.

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1. Christensen, C. L., "Designing Columns Against Bending and Compression," *Engineering News-Record*, 1928, V. 2, p. 127.

2. Albert, O., "Beam Method of Design for Eccentrically Loaded Columns," *Engineering News-Record*, 1943, V. 1, p. 365. Discussions by: Ellis, C. A., 1943, V. 1, p. 662; Borgwardt, W., 1943, V. 2, p. 249.

3. Hayden, A. G., "Combined Bending and Direct Stress," *Merriman and Wiggin's Civil Engineers Handbook*, p. 1048.

4. "Concrete Culverts and Conduits," Portland Cement Assn.

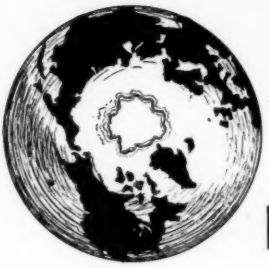
5. "ST-62 Combined Bending and Axial Load," Portland Cement Assn.

6. *Reinforced Concrete Design Handbook*, American Concrete Institute.

AUTHOR'S CLOSURE

The author is indebted to Lum for his attempt to demonstrate an easier procedure for computation of members subject to bending and direct stress.

His curve demonstrating the slight variance in the value of $A'_s + A_s$ for different values of f_s is of considerable interest. The writer is still of the opinion that his graph is as direct a way to solve these problems as it probably is possible to develop.



Letters from Readers*

BY WAY OF SYNOPSIS

L. S. MULLER discusses concentrated loads at the intersection of the beams on paneled ceilings.

F. L. FITZPATRICK and W. SERKIN describe experiments on the effect of storage of portland cement in 5-ply paper bags.

JOHN SIGGELKOW offers a chart showing recommended air contents as a function of the design strength of the concrete mix.

R. MERCIER describes a prestressed concrete bridge in Belgium

Prof. Holley stirred up a good deal of discussion on the teaching of reinforced concrete design. Two more replies are those by E. I. FIESENHEISER and MARVIN L. MASS.

Further questions on air entrainment are submitted by HENRY COMACK.

Bubble sizes in air-entraining concrete are discussed by ALBERT WEINER.

WALTER LUM and ANDRE RETI suggest methods of solving a foundation problem presented in the September JOURNAL.

Paneled Ceiling Loaded by Concentrated Loads at the Intersections of the Beams (LR 46-37)

L. S. MULLER, Chaikin and Nesher, Architects, Jerusalem, Israel, presents the solution to a problem of a grid of beams subjected to concentrated loads at the crossing points of the grids.

The approximate method given below, similar to that of Prof. Saliger's for a uniformly distributed load (*Der Eisenbeton*, A. Kröner, 1925), does not take into account the torsional resistance of the beams and the twisting moments in the corners. It is based on the consideration that the concentrated loads are divided between two intersecting beams in such a manner that the deflection of both beams at the point in question should be the same. Taking the intersection of the k (horizontal) and i (vertical) beam (Fig. 1) the distribution factor should be

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noted α_{ki}. This means that $\alpha_{ki}\ P_{ki}$ is the part of P_{ki} taken by the k beam of span L_x, and $(1 - \alpha_{ki})\ P_{ki}$ is the part of P_{ki} taken by the i beam of span L_y.

Writing the equality of deflections for every point of intersection we obtain as many simultaneous equations as the number of unknown distribution factors.

The cross section, the deflection of which we want to determine, may be to the right or to the left of the concentrated load in question (Fig. 2). Accordingly the deflection will be:

$$\left.\begin{aligned} &\text{if } x \geqq a\text{:}\quad f_x = \frac{Pab}{6\,E\,I\,L}\left[(L+a)\,(L-x) - \frac{(L-x)^3}{b}\right] \\ &\text{if } x \leqq a\text{:}\quad f_x = \frac{Pab}{6\,E\,I\,L}\left[(L+b)\,x - \frac{x^3}{a}\right] \end{aligned}\right\} \qquad (1)$$

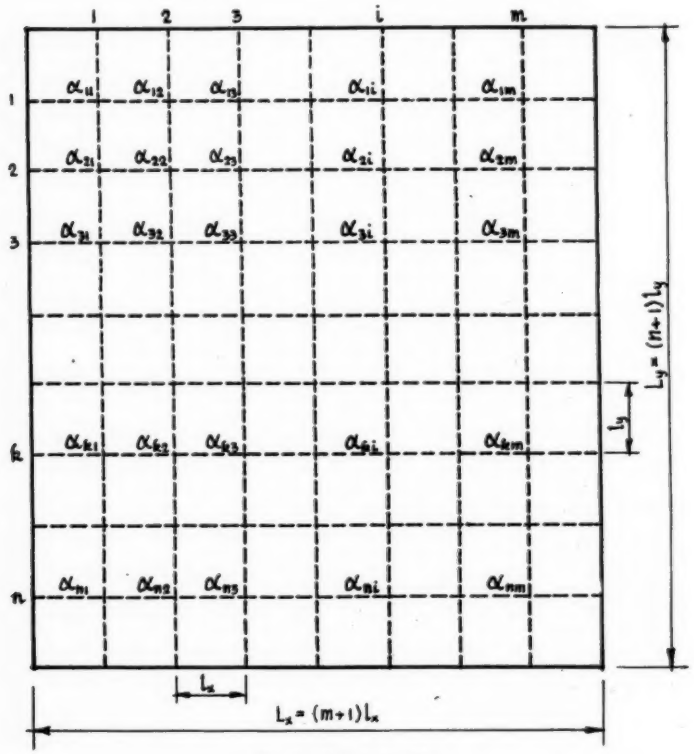


Fig. 1—Paneled ceiling

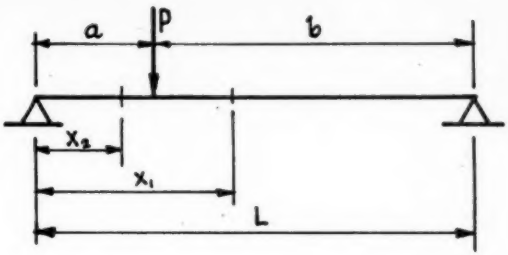


Fig. 2—Position of a concentrated load

If according to Fig. 1 $\quad l_x = \dfrac{L_x}{m+1}$

and $\quad l_y = \dfrac{L_y}{n+1}$

assuming further that the moment of inertia and E is the same for every beam, the deflection of the k beam may be written

$$f_{ki} = \frac{\alpha_{k1} P_{k1}}{6 E I L_x} \cdot \frac{L_x}{m+1} \cdot \frac{m L_x}{m+1} \left[\left(L_x + \frac{L_x}{m+1} \right) \left(L_x - \frac{i L_x}{m+1} \right) - \frac{\left(L_x - \frac{i L_x}{m+1} \right)^3}{\frac{m L_x}{m+1}} \right] + \ldots . + \frac{\alpha_{ki} P_{ki}}{6 E I L_x} \cdot \frac{i L_x}{m+1} \cdot \frac{(m-i+1) L_x}{m+1}$$

$$\left[\left(L_x + \frac{i L_x}{m+1} \right) \left(L_x - \frac{i L_x}{m+1} \right) - \frac{\left(L_x - \frac{i L_x}{m+1} \right)^3}{\frac{m-i+1}{m+1} L_x} \right] + \frac{\alpha_{ki+1} P_{ki+1}}{6 E I L_x} \cdot$$

$$\frac{(i+1) L_x}{m+1} \cdot \frac{(m-i) L_x}{m+1} \left[\left(L_x + \frac{(m-i) L_x}{m+1} \right) \frac{i L_x}{m+1} - \frac{\left(\frac{i L_x}{m+1} \right)^3}{\frac{i+1}{m+1} L_x} \right] + \ldots . +$$

$$\frac{\alpha_{km} P_{km}}{6 E I L_x} \cdot \frac{m L_x}{m+1} \cdot \frac{L_x}{m+1} \left[\left(L_x + \frac{L_x}{m+1} \right) \frac{i L_x}{m+1} - \frac{\left(\frac{i L_x}{m+1} \right)^3}{\frac{m L_x}{m+1}} \right]$$

Excepting $(m + 1)$ and L_x:

$$f_{ki} = \frac{\alpha_{k1} P_{k1} L_x^3}{6 E I (m+1)^2} m \left[\frac{(m+2)(m-i+1)}{(m+1)^2} - \frac{(m-i+1)^2}{m (m+1)^2} \right] + \ldots . +$$

$$\frac{\alpha_{ki}P_{ki}L_x^3}{6\,E\,I\,(m+1)^2}\,i\,(m-i+1)\left[\frac{(m+i+1)\,(m-i+1)}{(m+1)^2}-\frac{(m-i+1)^3}{(m-i+1)\,(m+1)^2}\right]+\frac{\alpha_{ki+1}P_{ki+1}L_x^3}{6\,E\,I\,(m+1)^2}\,(i+1)\,(m-i)\left[\frac{(2m-i+1)\,i}{(m+1)^2}-\frac{i^3}{(i+1)\,(m+1)^2}\right]+\ldots.+\frac{\alpha_{km}P_{km}L_x^3}{6\,E\,I\,(m+1)^2}\,m\left[\frac{(m+2)\,i}{(m+1)^2}-\frac{i^3}{m\,(m+1)^2}\right]$$

This may be written also in the form:

$$f_{ki}=\frac{\alpha_{k1}P_{k1}L_x^3}{6\,E\,I\,(m+1)^4}\left[m\,(m+2)\,(m-i+1)-(m-i+1)^3\right]+\cdots+\frac{\alpha_{ki}P_{ki}L_x^3}{6\,E\,I\,(m+1)^4}\,i\left[(m+i+1)\,(m-i+1)^2-(m-i+1)^3\right]+\frac{\alpha_{ki+1}\,P_{ki+1}\,L_x^3}{6\,E\,I\,(m+1)^4}\,(m-i)\left[i\,(i+1)\,(2m-i+1)-i^3\right]+\cdots+\frac{\alpha_{km}P_{km}L_x^3}{6\,E\,I\,(m+1)^4}\left[m\,(m+2)\,i-i^3\right] \quad \ldots\ldots (2)$$

Or generally:

$$f_{ki}=\frac{l_x^3}{6\,E\,I\,(m+1)}\left\{\sum_{z=1}^{i}\alpha_{kz}\,P_{kz}\,z\left[(m+1-z)\,(m+1+z)\,(m+1-i)-(m+1-i)^3\right]+\sum_{z=i+1}^{m}\alpha_{kz}P_{kz}\,i\,(m+1-z)\left[2z\,(m+1)-z^2-i^2\right]\right\} \quad \ldots (3)$$

The deflection of the same point expressed from the i beam:

$$f_{ki}=\frac{l_y^3}{6\,E\,I\,(n+1)}\left\{\sum_{z=1}^{k}(1-\alpha_{zi})\,P_{zi}\,z\left[(n+1-z)\,(n+1+z)\,(n+1-k)-(n+1-k)^3\right]+\sum_{z=k+1}^{n}(1-\alpha_{zi})P_{zi}\,k\,(n+1-z)\left[2z\,(n+1)-z^2-k^2\right]\right\} \quad \ldots (4)$$

Writing the equality of Eq. (3) and (4) for every point of intersection we obtain as many equations as the number of the distribution factors.

The number of equations will be considerably reduced if the forces are equal and the distribution factors are symmetrically placed at the center of the beams.

Example—For the paneled ceiling shown in Fig. 3 all the forces are equal, 10,000 lb.

$$L_x = 4 \times 5 \text{ ft} = 20 \text{ ft}$$
$$L_y = 5 \times 5 \text{ ft} = 25 \text{ ft}$$
$$l_x = l_y = 5 \text{ ft}$$

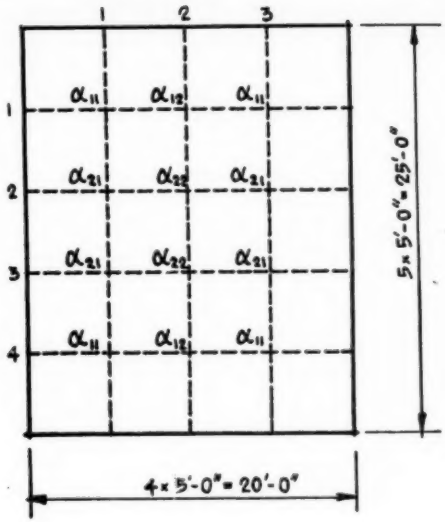


Fig. 3—Example problem for paneled ceiling loads of 10,000 lb

Omitting P, l_x, l_y, and $6\,E\,I$:

$$\tfrac{1}{4}\{\alpha_{11}[3\times5\times3-27]+\alpha_{12}\,2[4\times4-4-1]+\alpha_{11}[6\times4-9-1]\}=$$
$$\tfrac{1}{5}\{(1-\alpha_{11})[4\times6\times4-64]+(1-\alpha_{21})\,3[4\times5-4-1]+(1-\alpha_{21})\,2$$
$$[6\times5-9-1]+(1-\alpha_{11})[8\times5-16-1]\}$$
$$\tfrac{1}{4}\{\alpha_{11}[3\times5\times2-8]+\alpha_{12}\,2[2\times6\times2-8]+\alpha_{11}\,2[6\times4-9-4]\}=$$
$$\tfrac{1}{5}\{(1-\alpha_{12})[4\times6\times4-64]+(1-\alpha_{22})\,3[4\times5-4-1]+(1-\alpha_{22})\,2$$
$$[6\times5-9-1]+(1-\alpha_{12})[8\times5-16-1]\}$$
$$\tfrac{1}{4}\{\alpha_{21}[3\times5\times3-27]+\alpha_{22}\,2[4\times4-4-1]+\alpha_{21}[6\times4-9-1]\}=$$
$$\tfrac{1}{5}\{(1-\alpha_{11})[4\times6\times3-27]+(1-\alpha_{21})\,2[3\times7\times3-27]+(1-\alpha_{21})$$
$$2\times2[6\times5-9-4]+(1-\alpha_{11})\,2[8\times5-16-4]\}$$
$$\tfrac{1}{4}\{\alpha_{21}[3\times5\times2-8]+\alpha_{22}\,2[2\times6\times2-8]+\alpha_{21}\,2[6\times4-9-4]=$$
$$\tfrac{1}{5}\{(1-\alpha_{12})[4\times6\times3-27]+(1-\alpha_{22})\,2[3\times7\times3-27]+(1-\alpha_{22})$$
$$2\times2[6\times5-9-4]+(1-\alpha_{12})\,2[8\times5-16-4]\}$$

$$95\,\alpha_{11}+27.5\,\alpha_{12}+85\,\alpha_{21}=140$$
$$55\,\alpha_{11}+95\,\alpha_{12}+85\,\alpha_{22}=140$$
$$85\,\alpha_{11}+180\,\alpha_{21}+27.5\,\alpha_{22}=225$$
$$85\,\alpha_{12}+55\,\alpha_{21}+180\,\alpha_{22}=225$$

$$\alpha_{11}=0.687 \qquad \alpha_{21}=0.790$$
$$\alpha_{12}=0.285 \qquad \alpha_{22}=0.874$$

The loads on the beams will therefore be as follows:

Direction x: Beams 1 and 4: 6870 lb, 2850 lb, 6870 lb.
2 and 3: 7900 lb, 8740 lb, 7900 lb.

Direction y: Beams 1 and 3: 3130 lb, 2100 lb, 2100 lb, 3130 lb.
Beam 2: 7150 lb, 1260 lb, 1260 lb, 7150 lb.

Storage of Portland Cement in 5-Ply Paper Bags (LR 46-38)

F. L. Fitzpatrick and W. Serkin, Rocla Limited, Melbourne, Australia, describe experiments to determine the effect on cement strength of covered but not air conditioned storage.

A series of experiments was made to study the effect of covered, but not air conditioned, storage on the tensile strength of 3:1 mortar. Six 5-ply paper bags of cement were taken at random from a carload consignment.

Bag. No. 1 was opened immediately and tested. Then it was closed again and carefully wrapped in polyvinyl cloth as an additional protection against hygroscopic accretion. This bag of cement was stored in its wrappings and tested again after six months. Of the remaining five bags, which were stored in a dry room in the laboratory, one was opened and tested every month, the interval between tests being 28 days. These remaining five bags of cement were not wrapped in polyvinyl cloth. The storage room was not air conditioned, and so conformed more or less to the fluctuations of humidity and temperature of the Melbourne winter, spring and early summer seasons.

The tensile strengths obtained in these experiments, using the 3:1 mortar, are shown in Table 1.

TABLE 1—TENSILE STRENGTH OF 3:1 MORTAR*

Period of storage in bag, months	Bag No.	Tensile strength of test briquettes, psi		
		7 days water storage	28 days water storage	Stored 1 day damp air, 6 days water, 21 days air
–	1	364.0	399.0	578.0
6	1	344.0	349.0	520.5
1	2	261.5	382.5	547.0
2	3	266.5	359.0	489.0
3	4	258.0	303.5	479.0
4	5	249.0	296.0	443.0
5	6	248.0	331.5	472.0

*Proportions—3 standard sand: 1 cement: 0.4 water.

The total reduction in tensile strength of the cement in bag No. 1, stored for 6 months in a polyvinyl cloth wrapping, was: at 7 days water storage, 5.5 percent; 28 days water storage, 12.5 percent; and at 28 days combined, 9.9 percent.

The cement stored in unwrapped bags dropped within 1 month of storage as follows: briquette samples stored 7 days in water, 31.5 percent; briquette samples stored 28 days in water, 25.8 percent; and briquette samples stored 28 days combined 23.0 percent.

Bag No. 6 seemed to have contained a cement with a generally more favorable 28-day strength, though all were taken at random from the same consignment.

Discussing these results with the manufacturers of the cement, we were advised of somewhat similar experiments in which cement was stored in bags in a shed such as a contractor would use on a construction project and yielded results as shown in Table 2.

TABLE 2—DECREASES IN COMPRESSIVE STRENGTH OF CONCRETE*

Storage period	Compressive strength at 28 days as percent of strength of cement as received
As received	100
3 months	80
6 months	72
12 months	60
24 months	46

*Proportions—1 part cement to 5 parts aggregate by weight.

Their tests determined the strengths at later ages of concretes made from cement which had been stored in paper bags prior to use. It was found that some of the lost strength was recovered by long storage of the concrete samples. For example, when cement stored for 6 months in paper bags was used, 1:5 concrete samples yielded results as shown in Table 3.

TABLE 3—STRENGTH OF CONCRETE SAMPLES

Age of specimens (water storage)	Strength as percent of strength of similar concrete made with cement of same batch
7 days	73
28 days	75
6 months	84

SUMMARY

The tests indicated a slow deterioration of cement when stored in paper bags in ordinary covered storage. The rate of loss of strength is about 4 to 5 percent per month of storage in paper bags before use.

The Air Quantity in the Design of Air-Entraining Concrete
(LR 46-39)

JOHN SIGGELKOW, Assistant Civil Engineer, Dept. of Public Works, Westchester County, N. Y., presents a chart of recommended air contents for air-entraining concrete.

In the design of air-entraining concrete mixes, one of the basic elements is the amount of entrained air necessary or desirable.

How much should this be? Should it be the same for all mixes? Should it vary between gravel and broken stone concretes, between stone gradings of different top sizes?

In the literature concerning air-entraining concrete, investigators have been generally satisfied with the statement that the air content should be "between 3 and 6 percent" of the concrete unit. In the present development of this type of construction, as concerns actual field practice, this 3 to 6 percent is perhaps close enough.

But in our ability to meet tolerances we move forward with time, and quite likely this question of air content will require closer scrutiny, particularly for large jobs of mass concrete where cement is sparingly used.

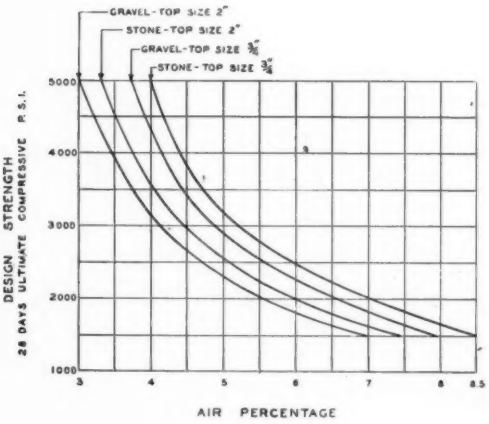


Fig. 1—Recommended air content of air-entraining concrete

The writer has devoted some study to this question, and as a result presents a chart of recommended air contents as a function of the design strength of the concrete mix (Fig. 1). While this chart is believed to be a definite improvement over the 3 to 6 percent requirement, the writer feels that the limitations on his study permit him to present the chart as tentative only.

He hopes, however, that its presentation may bring forth sufficient discussion to improve it to the point where it may be a useful tool in mix design.

Prestressed Concrete Bridge (LR 46-40)

R. MERCIER, Director, Societe Belge des Betons, Brussels, Belgium, describes a prestressed bridge recently completed in Belgium.

We recently constructed a prestressed concrete highway bridge on the Brussels-Ostende highway at Ghent, Belgium (Fig. 1). The bridge has a 95 ft span and is 63 ft wide (two sidewalks of 10 ft and a roadway of 43 ft). It also carries two streetcar railways with rolling loads of 30 tons.

There are 35 main girders, assembled together with lateral prestressing, with a depth of only 41 in. at the middle and 36 in. at the ends. The high tensile strength steel wire reinforcement was 9/32 in. in diameter. The concrete used for the girders totaled 630 cu yd and the weight of the steel wire reinforcement for the whole bridge was 63,000 lb, or 100 lb per cu yd.

The cost of the main girders only, including forms, concrete, steel wires, anchorage devices, prestressing and erecting, was $36,000 (at the rate of 50 Belgium francs to one dollar). The bid price for the whole job (demolition, foundations, construction of abutments, bridge and pavement, architectural devices and junction improvements) was about $300,000.

More information on the bridge will soon be published in the *Annales des Travaux Publics de Belgique.*

Fig. 1—Prestressed highway bridge, Ghent, Belgium

Teaching Reinforced Concrete Design—What Should We Emphasize? (LR 45-21)*

Prof. E. I. FIESENHEISER, Head of Structural Division, Dept. of Civil Engineering, Illinois Institute of Technology, Chicago, Ill., continues the discussion started in the June JOURNAL.

Our teaching of reinforced concrete at Illinois Institute of Technology is designed to provide the student with a knowledge of ordinary theory and its application to design practice. The courses are always taught by men who have had practical experience in this field of work, hence the emphasis comes from the instructor's knowledge of what is needed in the design office. Abstraction is avoided by presenting a theory and then immediately applying this theory to a practical problem. Thus the student senses at once the necessity of learning the theory in order to be able to do the work. Furthermore, the subject is so broad that there is no time for explorations of any but tried and accepted theories.

Regarding emphasis, experience with beginning students has shown that they do not easily distinguish between analysis and design. Unless this distinction is clearly in mind, confusion results. We therefore provide them with the following chart which lists the characteristics of the two types of problems:

Two types of problems occur in the study of reinforced concrete structures. One type is the checking or stress analysis of a *given* design to determine whether or not it is safe, or to determine what forces or moments may safely be applied. The other type is the *design* of a member for given working stresses. Before beginning work it is *always* necessary to classify the problem and to decide whether it belongs in the stress analysis or in the design category. To aid in making this classification, the following characteristics for each type of problem are listed:

Checking or stress analysis type of problem	*Design type of problem*
Dimensions of the cross section and location of reinforcing bars are given.	Dimensions of cross section are to be determined unless member is of limited size.
Area of steel or size and number of bars are given.	Total area of steel, size and number of bars are to be determined.
Neutral axis of the cross section *must* be determined by *solving an equation.* This will be a linear equation for tee-beams, a quadratic for rectangular beams and a cubic equation when both bending and direct stress are involved.	For an economical or "balanced" design the neutral axis of the cross section will be located from similar triangles using working stresses f_c and $\frac{f_s}{n}$.
The stresses for a given moment or the allowable moment for given working stresses are to be determined.	The design must be completely shown by use of a neat sketch drawn to scale. Dimensions and size and spacing of bars must be shown. *Use no decimals on design sketches.*

*See also ACI JOURNAL, June 1949, *Proc.* V. 45, p. 745; Sept. 1949, *Proc.* V. 46, p. 67 and Oct. 1949, *Proc.* V. 46, p. 148.

Reinforced concrete is taught in two separate courses. The beginning course consists of two lecture hours per week with homework of practical problems in stress analysis and design of beams, slabs, columns, and girders. Emphasis is placed upon the fundamental concepts, using the stress diagram and the transformed area of steel. The student is taught to derive the fundamental relationships without the use of formulas. Continuity is handled by use of ACI moment coefficients in this course.

The beginning course is supplemented by work in the concrete laboratory, where the student is introduced to the actual manufacture of concrete. Here he should learn what good concrete and a workable mix look like. Proportioning methods, water-cement ratio, admixtures, tests of aggregate, standard cylinder tests, and flexure tests of plain and reinforced beams are studied.

This work is followed by another course in reinforced concrete design. This course consists of two lecture hours per week plus a three-hour computing period. Here the student learns how to design actual structures with their component parts. For use in the design of continuous building frames the moment distribution method is taught and applied. Included in the course are reinforced concrete retaining walls, highway slab and tee-beam girder bridges, pile foundations, continuous girders, and finally a building frame design problem which utilizes moment distribution. Emphasis is placed upon the preparation of neat and clear design drawings containing *all* the information required for building the structure. The differences between design and placing drawings are learned and the *ACI Manual of Standard Practice for Detailing Reinforced Concrete Structures* is one of the required textbooks.

Prof. MARVIN L. MASS, Dept. of Civil and Architectural Engineering, University of Colorado, Boulder, Colo., adds "To Whom?" to the question of "What Should We Emphasize?"

Prof. M. J. Holley poses a very interesting question in his letter in the June 1949 JOURNAL, "Teaching Reinforced Concrete Design—What Should We Emphasize?" I certainly agree with him that the courses offered by our engineering schools can be improved, and that these changes in course content or teaching methods should reflect the experiences and requirements of men active in the profession—both those who design and those who employ designers. However, I believe that we must also consider *who* we are training. We need only take a look at the jobs the graduates obtain and notice that the types of jobs are very diversified, and the smallest percentage of the students get, or

even look for, design jobs. Therefore, before we can decide what we should emphasize, we must look into the students we are to train.

If you canvas the freshman civil engineering students you will find that a majority of them want to be structural engineers. If you canvas the seniors most of them have changed their minds, and if you look at the statistics of what jobs the alumni of almost any school hold, I believe that only a comparatively small percentage have actually gone into the design field. For example, only 30 percent of the recent graduating class at the University of Colorado have jobs in design offices or jobs that will lead to design positions. In ten years it is likely that a good number of this 30 percent will have drifted from the design field. Of my present undergraduate reinforced concrete design class I find only four out of sixty students plan on being designers. Most of the others readily admitted that they were taking the course simply because it is required for a civil engineering degree. It seems, therefore, that we should take these facts into consideration in planning courses. I believe that the curriculum of most schools has remained unchanged for a number of years. This does not mean that there has been no improvement in the courses because most course material has been revised to keep up to date with the latest scientific information, but the general category of subjects has not been changed. (There are some exceptions in the curricula now being offered by some schools as options to train construction men, structural men, etc. However, most of the options still require all the undergraduates to take all of the reinforced concrete offered.)

Let us look at each of the civil engineering student groups to be trained: (1) the student who aims at being a structural engineer; (2) the student who aims at something other than being a structural engineer; (3) the architect and architectural engineer and (4) the student who does not know.

Each of the above groups of students would naturally have different ideas on what they would want to obtain from a course in reinforced concrete. Yet the time allotted in the undergraduate curriculum for the study of reinforced concrete is only sufficient to teach the same course content to all four groups.

Irrespective of the time available for teaching reinforced concrete in the undergraduate curriculum, it seems to me that the most complete set of objectives (which includes Prof. Holley's suggestion) are the following seven points:

1. To familiarize the student with the nature of a concrete building. How is a concrete building constructed? How does it differ from a steel building? What are the limitations of a concrete building? What are types of construction, flat slab, beam and girder, etc.?

2. To familiarize, only superficially, the student with the currently accepted theories of reinforced concrete such as the straight line theory.

3. To train the student thoroughly in the use of the accepted theories.

4. To train the student in the use of the short cut methods and to become proficient in the use of the tables.

5. To inform and train the student in special problems dealing with indeterminate structures, primarily in the methods of design, analysis, and construction practices.

6. To train the student in the presentation of reinforced concrete design calculations and details.

7. To inform the student that new methods are being devised such as the plastic theory and prestressed concrete.

The question now arises how many of these objectives should be taught to *all* students in the time allotted in an undergraduate curriculum? Since there is usually only a minimum time available, *all* students might be required to take a reinforced concrete course consisting only of objectives number 1 and 2 and possibly a smattering of number 7. This would acquaint everyone with some of the underlying principles and would be a good foundation for those students who wanted to delve into the subject more thoroughly either in school, on their own, or on the job. If the teaching staff is available, it would be helpful to students and future employers to offer the other four objectives as electives in advanced undergraduate or beginning graduate courses.

It is hoped this broad background would inspire more civil engineering students to follow the field of structural design, as the present method of teaching tedious details without the proper understanding of the action of a reinforced concrete structure tends to bore and confuse the student and cause them to seek other fields.

The results of this discussion is little different from the points made by Prof. Holley and Mr. Andrews (September JOURNAL). This is meant merely to add another argument for "liberalizing" the undergraduate reinforced concrete curriculum.

What is the Difference Between Air Entrained in Ordinary Concrete and That Resulting from an Air-Entraining Agent? (LR 45-22)*

HENRY COMACK, Flushing, N. Y., desires more evidence to prove that air benefits concrete.

In the June JOURNAL, Anton A. Anjel posed the question: "What is the difference between air entrained in ordinary concrete and that resulting from an air-entraining agent?" Like Mr. Anjel, I had experienced as high as 4 percent air with no air-entraining admixture. I can only add

*See also ACI JOURNAL, June 1949, *Proc.* V. 45, p. 747; Sept. 1949, *Proc.* V. 46, p. 68 and Nov. 1949, *Proc.* V. 46, p. 227.

to this problem, and produce several enigmas along similar lines, trusting that someone more expert in air entrainment will clarify the problems in a future JOURNAL.

We are led to believe that there is "good" and "bad" air in concrete. Bad air is produced without the benefit of an air-entraining agent, or with an agent in excess of an optimum amount. It is said that air is bad when it is contained in pores above a microscopic size. How the air creates increased durability is said to be caused by the "room" for expansion and contraction, thereby creating "elasticity." In my studies of physics, chemistry and biology, I recall that water can enter small apertures of microscopic size and this movement is known as capillary action. Is it a "special" air that can exclude water in concrete? Or, if the water enters, does the texture permit freezing and thawing within the voids without causing noticeable cracking, scaling, etc.?

During the technical program of the 1949 ACI Convention one of the speakers was asked to explain how air benefits concrete when a good deal of it gradually disappears during the setting-up period. It was simply answered by the statement that it is the final amount that counts and that the initial amount of air determines the proper final amount.

There is no doubt that certain admixtures that cause the entrainment of air also create a more durable concrete. However, I am skeptical as to whether it is the entrained air that is the cause. I therefore would like to ask the following questions:

1. Is it possible that certain waterproofing chemicals are created by the admixtures when added to cement, and that air is a by-product and a convenient gage of quantity?
2. Is it possible that the increased workability created by such agents really produces a more durable end-product, not the air?
3. Is there any substantial scientific *evidence*, not theory, nor circumstantial evidence, that can convince me that it is the *air which benefits concrete?*

Since air entrainment is being used more and more, it is the responsibility of those sponsoring its use to answer many of the doubts regarding the theories explaining it.

Bubble Sizes in Air-Entraining Concrete (LR 46-24)*

ALBERT WEINER, Materials Engineer, North Atlantic Division, Corps of Engineers, Dept. of the Army, New York, N. Y., makes additional comments on bubble sizes.

Dr. Anderegg's comments concerning bubble size in concrete in the September JOURNAL are illuminating. It is regrettable that at this late date, and in spite of extensive research on the subject of air entrain-

* See also ACI JOURNAL, Sept. 1949, *Proc.* V. 46, p. 70.

ment, so little has been done on the subject of measuring and controlling air bubble size. The effect of concentrating our efforts on the presence and amount of air is similar to what would be the case if fine aggregate for concrete were specified without recourse to sieve analysis.

Another equally suggestive element in air entrainment is the distribution of bubbles in the mortar. The writer has observed concrete in which the voids had, for no apparent reason, concentrated on the surface of the coarse aggregate resulting in the aggregate almost literally floating in air. The mortar-aggregate interface had the appearance of pumice. Aside from the marked reduction in strength produced by such maldistribution of air, serious doubts concerning the durability of such "air-entraining" concrete could well be raised.

Perhaps some of the lack of information may be traced to the tediousness of measuring air bubble size. The camera lucida and intercept methods are accurate and well adapted to the problem but are rather slow. The writer has obtained average bubble diameters by counting the number of bubbles in an aliquot of plastic mortar and equating number and average individual volume to the total air content separately determined by gravimetric or other means. This method is best used where the maximum aggregate size is not too great, for the count is made by compressing the mortar between two glass plates set apart at a fixed distance. It is possible to direct a light beam through the assembly and make a fairly accurate count of the bubbles.

The question of bubble size and distribution measurement introduces the possibility of control in concrete. It is probable that bubble size is distributed according to a normal frequency distribution, but the effects of deviations from symmetry should be considered. Control of the average size of bubbles may be possible and advantageous. Use of means for air entrainment other than by reduction of surface tension may be fruitful. In this connection, preferred gradation of bubble sizes may be achieved through the use of graded aluminum powder, hydrogen peroxide catalyzed by catalase, or materials such as nitramide whose decomposition rate can be controlled.

Soil Pressure Under a Continuous Foundation Slab (LR 46-26)*

WALTER LUM, Cleveland, Ohio, suggests a reference to a solution of the problem presented in the September JOURNAL.

The problem involving a continuous flexible foundation slab under three or more column loads is ably solved by the late C. A. Ellis in his

*See also ACI JOURNAL, Sept. 1949, *Proc.* V. 46, p. 71.

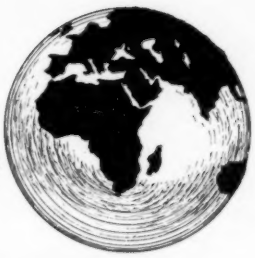
book *Essentials in the Theory of Framed Structures*, McGraw-Hill, 1922. The solution is based on the usual assumptions of the straight line elastic theory. Soil mechanics, however, may have a different theory, regarding the actual behavior of soil reactions.

ANDRE RETI, Civil Engineer, Caracas, Venezuela, also suggests a method of solution.

The problem of the common foundation for several columns with different loads and different spacings can be solved by means of the theory of the continuous beam resting on elastic foundation (See the book of Prof. Hetenyi, *Beams on Elastic Foundation*). Handbooks on reinforced concrete construction and foundations do not explain anything about this subject because the numerical calculus is not easy, especially when the span of the beam is limited and the loads are found near the ends.

A system for practical calculus which permits the solution of the more complicated cases without too much work was published in the October and November, 1949, issues of the French magazine *Travaux*.

Current Reviews*



of Significant Contributions in Foreign and Domestic Publications

Lining irrigation canals

Portland Cement Assn., Chicago, 1949. 42 pp.

To prevent seepage losses, to reduce the required size of canal, and to lower maintenance costs many irrigation canals are lined with concrete. This booklet describes and illustrates the use of portland cement concrete, pit-run concrete, shotcrete and cement mortar linings for this purpose. The tangible and additional benefits of canal lining are evaluated and compared with the cost of the lining. Charts are included for the design of canal cross sections. Specialized equipment for shaping the canal and applying the lining by several methods are shown.

Style manual for American Standards

American Standards Assn., New York, 28 pp. $1.00

A guide in the preparation and publication of American standards to help bring about greater uniformity in the style, format, and presentation of technical data, not only in the standards published by ASA but also in the standards published by sponsors. While, in conformance with its purpose, much of the material applies specifically to ASA publications, valuable hints of general interest are included. In this category, rules of spelling, punctuation, abbreviations, italics, use of numerals and preparation of bibliographies are of value to the technical writer.

Prefabricated units (Betonelementenbouw)

A. J. VAN WALRAVEN, *Cement* (Amsterdam), V. 1, No. 1-6, 1949, pp. 10-12, 53-57, 75-81

Reviewed by J. W. T. VAN ERP

A nitrate storage shed of the blast furnace plant in Ijmuiden, Netherlands, is capped by a 90-ft span three-hinged arch 80 ft high. Reinforced concrete was chosen over timber (danger of combustion in nitrate-dust atmosphere) and steel (nitrate brittleness due to invisible, but dangerous intercrystalline corrosion). Methods of protection of the concrete against this aggressive corrosive atmosphere are given. Details of construction were influenced by the particularly acute shortages of labor, timber and reinforcing steel in 1946-47. The efficient erection procedure of the trusses, avoiding all work at great height, is described.

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Review of current research

Engineering College Research Council (F. W. Dawson, State University of Iowa, Iowa City, Chairman). 186 pp. $1.75 postpaid

Over 4000 current college and university research projects in engineering subjects, representing expenditures of over $35,000,000, are listed by title. Entries from 82 educational institutions which hold membership in the Research Council describe the administrative policies for conducting engineering research and list the responsible personnel, research expenditures, short courses and conferences of special interest, and the titles of all engineering research studies currently active at each institution.

Use of the volume is facilitated by a breakdown of research projects according to the engineering departments involved, and by a complete index to research project subjects.

Gap graded aggregates in concrete

H. N. Walsh, *Commonwealth Engineer* (Australia), V. 33, No. 11, June 1949, pp. 465-470

A preliminary review of Abrams' fineness modulus methods of design is followed by a discussion of gap grading of aggregates. Three different ways of forming a gap while omitting the same size in each case are illustrated. From the curve of the control grading a gap grading curve is formed by drawing a horizontal line across the space representing the size to be omitted. Depending on the position of this line with respect to the control curve the author classifies the gradings as centered, high or low gap. The effects of these gradings on concrete mixes is then indicated.

Reference is made to the use of gaps in design of mixes in practice and the need for further investigation is pointed out.

New reinforced concrete frames support overhead wires for Dutch electric railroads (Nieuwe Draagconstructies van gewapend-beton voor de electrische bovenleiding der Nederlandsche Spoorwegen)

J. V. Zutphen, *Polytechnisch Tydschrift*, V. 4, No. 31-32, Aug. 11, 1949, pp. 459-462

Reviewed by Charles C. Zollman

This article describes briefly the casting in a factory and the erection and assembly at the site, of the standardized foundation blocks, the three types of poles and single type of cross-beam used to form all types of frames required to support the overhead wires of the electric railroad.

All the parts are of ordinary reinforced concrete. Dimensional drawings and photographs illustrate the description.

Chemical reactions of Indiana aggregate in disintegration of concrete

Floyd O. Slate, A.S.T.M. 1949 Preprint, No. 50, 8 pp.

Author's Summary

This study was made to determine whether or not chemical reactions of aggregates have played an important part in the disintegration of certain concrete pavements in Indiana. Previous field work had shown a positive correlation between certain types of disintegration and type of coarse aggregate used in the mix. In the present study, two general approaches were used: first, a series of widely accepted, empirical tests for chemical reactivity between cement and aggregate were performed (expansion bar tests, alkali-immersion pat tests, and alkali-reactivity "bomb" tests) on most of the major types of Indiana's limestone aggregates and some of the gravels—both good and bad. Second, several specific tests were made on the suspected reactants (chemical and petrographic analysis of aggregate, petrographic analysis and macroscopic examination of disintegrated concrete, and others). Considerable information was collected concerning chemical and mineralogical composition of several of Indiana's sedimentary rocks. For the aggregates studies, no appreciable chemical reaction was found which would cause disintegration of concrete.

Arch dam conserves water for Vail Ranch

EARL HAGADORN, *Civil Engineering*, V. 19, No. 10, Oct. 1949, pp. 36-37 Reviewed by MARY KRUMBOLTZ HURD

Recently completed Vail Dam on Temecula Creek, Riverside County, Calif., is a constant-angle structure containing 30,000 cu yd of concrete. Providing 50,000 acre-ft of storage at spillway level, the dam varies from 4½ ft to 15 ft in thickness with a maximum height of 150 ft. Because of excessive lean at the ends of the arches, 2-ft thick bracing walls were built for stability during construction and when the reservoir is empty. The separate spillway, which extends 520 ft from the upstream end of the south gravity section of the dam, will discharge 72,000 cfs or more than three times maximum flood recorded.

Aggregate from nearby stream beds ranging from pebbles to 6-in. cobbles was used with 4 sacks of low heat cement per cu yd to make concrete testing 4000 psi at 28 days. One-fourth lb Pozzolith per sack of cement was added to overcome dumping difficulties. Using truck and crane, over 500 cu yd of concrete per working day was placed from an automatic batching and mixing plant 300 ft downstream. A minor fault at the dam base was pressure grouted to increase bearing value and seal against percolation.

Concrete pile manufacture at the Port of Lannceston, Tasmania

ALLEN BENNETT, *Journal of the Institution of Engineers* (Australia), V. 21, No. 45, April-May, 1949 Reviewed by J. R. SHANK

Two thousand concrete piles for wharf construction were manufactured in a specially designed and constructed plant costing about £25,000 with costs per pile about 25 percent less then they would have been otherwise. The final cost per linear foot of pile was £4/12/9. The saving was three times the cost of the plant which is described in detail.

Piles were octagonal, 21 in. short diameter, carrying eight 1¼-in. round bars and ⅜-in. spiral steel at 3 in. pitch. Sixty-pound cart iron conical shoes with dowels cast into the shoe were used. These dowels were welded to the bent-in ends of half of the longitudinal bars. The conical section of pile and shoe was 18 in. long, half of which was shoe. Piles were from 48 to 80 ft long. Longitudinal bars were butt welded at all joints. The pitch of spirals was reduced to 2 in. in the 6 ft at both ends. Two in. pipe holes were left in the piles for handling bolts. Piles were side poured with 2 in. slump concrete using quartzite gravel and 7 bags of cement for cu yd producing 4400 psi at 28 days. Form vibrators were used. After stripping the forms, the piles were sprayed with concrete curing oil and left for 3 weeks before stacking.

The design of concrete mixes containing entrained air

THOMAS H. THORNBURN, A.S.T.M. 1949 Preprint, No. 49, 16 pp. AUTHOR'S SUMMARY

This investigation was undertaken to determine whether the mortar-voids method of concrete design originally suggested by A. N. Talbot and F. E. Richart could be adapted to the design of concrete mixes containing entrained air.

A preliminary study of the characteristics of mortar-voids curves obtained for mortars containing entrained air indicated that these curves cannot be issued as a reliable basis for the design of concrete containing entrained air.

A study of the strength of controlled mortar and concrete mixes containing entrained air indicated that within the range of strengths commonly used for pavement concrete a significant linear correlation exists between the strength at a given age and the cement-space ratio.

It is suggested that theoretical mortar-voids curves for mortars containing entrained air may be prepared from the experimentally determined curves for mortars

which do not contain entrained air. It is demonstrated how these theoretical curves may then be used to proportion a concrete mix of any desired strength provided the air entrained in the concrete corresponds to that assumed in the preparation of the theoretical design curves.

Tests on a series of laboratory mixes proportioned according to the suggested method indicated that the method is at least as accurate as the best methods used in the design of concrete mixes which do not contain entrained air.

A wetting-and-drying test for predicting cement-aggregate reaction

C. H. Scholer, A.S.T.M. 1949 Preprint, No. 46, 8 pp. Author's Summary

This paper presents a method of determining the probability of a cement-aggregate reaction in any mixture of cement, fine aggregate, and coarse aggregate. The test essentially consists of subjecting specimens of concrete, approximately 3 x 4 x 16 in. to exposure of alternate drying at a raised temperature and wetting immersed in water at room temperature, with one cycle being completed in 24 hours.

For the extremely rapid reactions that may be secured with unusually high-alkali cement and reactive aggregates, it is much faster than the commonly accepted mortar bar test in a sealed container. The test is much more sensitive in that many combinations which do not show reaction under the commonly accepted mortar bar test will show abnormal expansion in the wetting-and-drying test.

Whether or not all of these expansions are directly the result of cement-aggregate reaction may be questioned, but it is believed that the test is highly significant and that where appreciable expansions occur under this test, there is considerable danger of serious reaction in such concrete combinations. The test is peculiarly valuable in that it may be used with coarse aggregate combinations and also because it shows the significant effect which various coarse aggregates may have upon the so-called cement-aggregate reaction. Some of the data indicate that the commonly accepted alkali limitations may not be very significant, and probably other elements than alkali alone are potent factors in determining the extent of the cement-aggregate reaction.

Prestressed concrete (Voorgespannen Beton)

J. J. B. J. Bouvy, *Cement* (Amsterdam), V. 1, No. 1-2, 1949, pp. 14-19 Reviewed by J. W. T. van Erp

The article explains the principles of prestressing concrete, still too little known, and summarizes experiences with it in actual structures. Considerable savings in construction cost are possible in properly designed structures, particularly in Europe with its existing ratio between labor and material cost. In this country where that ratio is up to 2½ times as high, the same savings are not to be expected since they are in materials (particularly in long-span structures like bridges and roof trusses) while more labor is needed.

Ordinary reinforced concrete shows decided defects which, due to the ever increasing demands upon it, tend to become more and more conspicuous. A typical reinforced concrete member has a compressive zone where the stresses are taken up by the concrete, eventually aided by compressive reinforcement, and a tensile zone in which all stresses are taken up by the reinforcement. Except for protection against atmospheric influences, the concrete in the tensile zone, therefore, functions only to transfer forces to the reinforcement. In this concrete fails, since in the tensile zone it always shows a multitude of cracks which start to form when the steel stress exceeds 12,000 psi. As the concrete in the tensile zone has no actual part in taking the main forces, its bulk in a conventional structure wastes material and adds appreciably to the dead load. Cracking can be ameliorated by increasing the compressive strength of the concrete by improvement in the quality of the cement, improvement in the granulometric structure of the

aggregates, and reduction of the water-cement ratio by properly vibrating the poured concrete. These methods, however have little effect on the tensile strength. An improvement in the quality of the reinforcing steel worsens the situation. The use of high-strength steel means increased stresses, increased elongation and thus increased cracking in the tensile zone of the concrete. It therefore becomes imperative to set up an entirely new concept of the cooperation between concrete and steel.

The aim is to have the concrete with its excellent compressive characteristics take up only compressive stresses, with no tensile stresses occurring in it anywhere. This is possible by imparting initial compressive stresses in those parts of the member in which tensile stresses occur under load, with the restriction that the ultimate stress never exceeds the limit to which the material can be subjected for an infinite period. This is the principle of prestressed concrete.

A further elaboration reveals that the entire concrete section is used to take part in the main stresses. In the unloaded condition there will be a triangular compressive stress diagram for the entire concrete section with its maximum on the reinforced side. Under full loading this diagram is reversed and the maximum compressive stress occurs at the unreinforced side.

This principle is applied by post-stretching by applying the prestress after the concrete has been hardened, and by prestretching the steel before the concrete is poured.

There are many complicating factors; concrete has no distinct yield point so that under permanently applied precompression there is a certain amount of plastic flow which reduces the initially imparted tension in the steel. Shrinkage in the concrete and yield in the highly stressed steel often account for a reduction in the steel stress by up to 17,000 psi. This makes it imperative to use high strength steels. Ordinary mild steel is unsatisfactory as has been pointed out by Freyssinet.

A comparison of plain reinforced and prestressed concrete beams demonstrates the economy possible with this method of construction. For a 100-ft span and a uniform load of 20 kips per ft it was found that the amount of concrete was reduced by 64 percent, while the total cost of the steel reinforcement (assuming that the cost of the high strength steel used is 2.5 times that of ordinary reinforcing steel) is even more reduced, namely by 72 percent. These savings are greatest for structures of long span with heavy loads. For short spans large savings are obtained only through quantity prefabrication.

While many structures are of the post-stretched type, prestretching also is used. The prefabricated elements are poured in series so that the reinforcement can be continuous; the stretching being done from end anchorages 300-500 ft apart. The initial tensile stress in the steel decreases through elastic compression and because anchorage is needed at each end over which the bond stress can add up into the full steel stress. Several methods accomplish this by (1) giving the wire a series of nicks or deformations over its anchorage length (Switzerland), (2) undulating the wire, which undulations after the prestressing remain in a reduced form, hardly visible, but just enough to give the desired anchorage (France), and by (3) increasing the perimeter area by using thin wires (France and Germany).

Design and construction of reinforced concrete bridges

A. W. LEGAT, G. DUNN and W. A. FAIRHURST, Concrete Publications, Ltd. (London), 528 pp. $6.50 postpaid

Reviewed by LEONARD C. HOLLISTER

In reading this book, one is reminded of the work by J. A. L. Waddell, on "Bridge Engineering." The authors, like Waddell, take the reader by the hand and explain to him step by step the various problems to be encountered in the construction of a reinforced concrete bridge. They start with the survey and site investigation, passing

on to the selection of type, and on through the design, estimating and construction of the bridge. With reference to the ideal bridge the authors say, ". . . perhaps the criterion of bridge construction on most sites, so far as the roadway is concerned, is that one should be able to pass over it without having it impressed on one that a bridge is there."

Four chapters deal with the design of various types of structures. In connection with slab and girder bridges there is given the Ministry of Transport standard loading which is interesting in comparison to the loading used by the American Association of State Highway Officials. The Ministry of Transport live load is a uniform load varying from 2500 lb per sq ft for a 3-ft span to 70 lb per sq ft for spans 250 ft and greater. In addition to the uniform load there also is applied a knife edge load across the entire width of roadway amounting to 2700 lb per ft of width. For the average bridge, then, the Ministry of Transport loading is considerably heavier than the A. A. S.H.O. loading.

It is possible that this difference in loading accounts for the feeling gained, as one looks over the plans and pictures in the book, that the bridges designed by the authors are much heavier and more massive than the usual highway bridge in this country.

Two rules are set forth for the transverse distribution of concentrated loads in connection with the design of reinforced concrete slabs. One is Slater's formula, based on experimental data, and the other is that proposed by Pigeaud and derived by the use of partial differential equations. Twelve different graphs are given for the use of Pigeaud's method. It is interesting to note that both of these formulas for transverse distribution of concentrated loads provide for a rather high percentage of transverse reinforcing and more than would be required by A.A.S.H.O. specifications.

In the chapter devoted to the design of reinforced concrete members subjected to combined thrust and bending several graphs are presented to help the designing engineer compute the stresses in members subjected to combined thrust and bending.

For the design of reinforced concrete arches of the three-hinged, fixed and bowstring types no exact method is given for the calculation of fixed arch stresses, but useful data is provided in the form of tables and formulas for the first trial design of the arch. The authors believe that after one develops a facility for the use of the tables and formulas ordinary arch designs can be laid out which will require but little adjustments after the final check analysis.

For foundations and the design of footings for various types of loads under various foundation conditions there are many detailed drawings and photographs of various footings for arches and other structures. On the strengthening, widening and realignment of existing bridges the authors describe various types of widening and strengthening jobs and include a few paragraphs on determining the strength of existing bridges by test loads.

Other chapters cover office practice, estimating, construction methods and equipment, forms and false work, ordering and handling of reinforcing steel, mixing, placing and curing of concrete, and the duties of the resident engineer.

Special features of reinforced concrete bridge construction discussed include railway bridges, culverts and subways, precast concrete bridges, prestressed concrete bridges, jacking arches, temporary and permanent concrete hinges, drainage and waterproofing, wind pressures on reinforced concrete bridges, and testing and maintenance.

An appendix deals with conditions of contract and gives a sample contract specification.

All in all, the book is very interestingly prepared and should be of great value to any engineer in the British Isle, and of interest and value to engineers of other countries engaged in bridge design and construction.

It is regretted that the authors did not devote more space to prestressed reinforced concrete members for bridges. It is believed that the English are somewhat more

advanced in this field of reinforced concrete bridge construction than are we in the United States. The authors, therefore, might have considerably enhanced the value of their book to bridge engineers in this country if they had devoted more space to prestressed concrete bridge members.

The efficiency of concrete mixing plant

L. J. MURDOCK, *Institution of Civil Engineers Journal*, V. 31, No. 1, 1948, pp. 56-81 BUILDING SCIENCE ABSTRACTS

An account is given of investigations made to determine the efficiency of concrete batching and mixing plant as judged by (1) quality and uniformity of concrete produced, (2) speed of operation, (3) freedom from mechanical troubles. The plant examined comprised: (a) two types of cement-weighing machines working in conjunction with overhead storage hoppers, (b) various methods of batching aggregate by volume and by weight, (c) three types of water-measuring tanks and supply systems, (d) various types of power mixers, including an American mixer with a two-compartment drum, (e) continuous mixers, (f) two types of rotating drum mixer of 10 cu ft capacity, (g) a laboratory mixer of the rotating pan type. It is concluded that the accuracy of weighing-machines and water-measuring devices should always be verified at frequent intervals, and that concrete mixers do not necessarily produce uniform concrete. Weight-batching machines working on the beam and jockey-weight principle appear the most satisfactory, although clogging of the knife-edges may occur. When used with small concrete mixers, weighing machines could advantageously operate in conjunction with the loading hopper, so as to avoid the necessity of a separate weigh-bucket. The main problems lie in releasing the loading hopper from other support during weighing, and in providing against shock. With careful maintenance, vertical measuring-tanks are reasonably accurate when calibrated properly. In determining the efficiency of concrete mixers, many factors are involved, including "sticking," which becomes serious in "dry" mixes, but the "efficiency number" method of assessment, while arbitrary, gives a fair indication of mix uniformity. In both small and large rotating drum mixers there is a general lack of interchange of materials from one end of the drum to the other. Continuous mixers and pan mixers also show lack of uniformity, but there is some evidence that concrete shows less segregation when discharged from tilting drum mixers. Until the design of mixers has been improved greater uniformity in concrete may be attained by paying attention to the sequence in which the various materials are placed in the loading hopper, and by discharging into an intermediate hopper before transport to the site, thus assuring some degree of premixing and post-mixing. The speed of the mixer should be regulated to that recommended by the makers.

PERIODICALS REGULARLY SCANNED FOR REVIEWS

The many requests for information concerning the source material of reviews appearing in the "Current Reviews" section indicate the desirability of publishing a list of those periodicals being regularly scanned for articles dealing with the field of concrete. Included also are the publisher's addresses for convenience should correspondence with them be desired.

Betong	Brunkebergstorg 15, Stockholm, Sweden
Betong Og Jernbeton	15 Ridebanvang, Gentofle, Denmark
Betonstein Zeitung	Bingerstrasse 14, Wiesbaden, Frankfurt Am Main, Germany

Bulletin, A.S.T.M.	1916 Race St., Philadelphia 3, Pa.
Bulletin, Institution of Civil Engineers in Ireland	35 Dawson St., Dublin, Ireland
Bulletin, University of Illinois	Urbana, Ill.
Cement	Sarphatistraat 54, Amsterdam C, Netherlands
Cement and Lime Manufacture	14 Dartmouth St., London (Westminster S.W.1), England
Cement och Betong	Malmo, Sweden
Cemento Portland	Calle San Martin 1137, Buenos Aires, Argentina
Civil Engineer Corps Bulletin	Bureau of Yards and Docks, U.S.N., Washington 25, D. C.
Civil Engineering	33 W. 39th St., New York 18, N. Y.
Commonwealth Engineer, The	349 Collins St., Melbourne, C. 1, Australia
Concrete	400 W. Madison St., Chicago 6, Ill.
Concrete and Constructional Engineering	14 Dartmouth St., Westminster S.W.1, England
Constructional Review	14 Spring St , Sidney, N.S.W., Australia
Crushed Stone Journal, The	1415 Elliot Place, N.W., Washington 7, D. C.
Current Road Problems	Highway Research Board, 2101 Constitution Ave., Washington 25, D. C
Engineering Experiment Station News	Ohio State University, Columbus, Ohio
Engineering News-Record	330 W. 42nd St., New York 18, N. Y.
Hormigon Elastico	Lavalle 1171, 5°, Buenos Aires, Argentina
Handlingar (Proceedings)	Ingeniors Vetenskaps, Akademien, Stockholm, Sweden
Ingenioren	Copenhagen, Denmark
Journal of the American Ceramic Society	2525 N. High St., Columbus 2, Ohio
Magazine of Concrete Research	52 Grosvenor Gardens, London S.W.1, England
Osterrichische Bauzeitschrift	Vienna, Austria
Polytechnisch Tydschrift	Haarlem, Netherlands
Public Roads	Highway Research Board, 2102 Constitution Ave., Washington 25, D. C.
Reinforced Concrete Review, The	94-98 Petty France, London S.W.1, England
Revista de Obras Sanitarias de la Nation	Charcas 1840, Buenos Aires, Argentina
Revue des Materiaux	75 Rue Lafayette, Paris 9, France
Rock Products	309 W. Jackson Blvd., Chicago 6, Ill.
Structural Engineer, The	2 Bream Bldgs., London E.C.4, England
Teknisk Tidskrift	Kungsgatan 37, Stockholm C, Sweden
Teknisk-Ukeblad	Oslo, Norway
Travaux	6 Ave. Pierre 1st de Serbie, Paris 16, France